JULY 1956

Journal

VOL. 48 • NO. 7

AMERICAN WATER WORKS ASSOCIATION

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Journal

AMERICAN WATER WORKS ASSOCIATION

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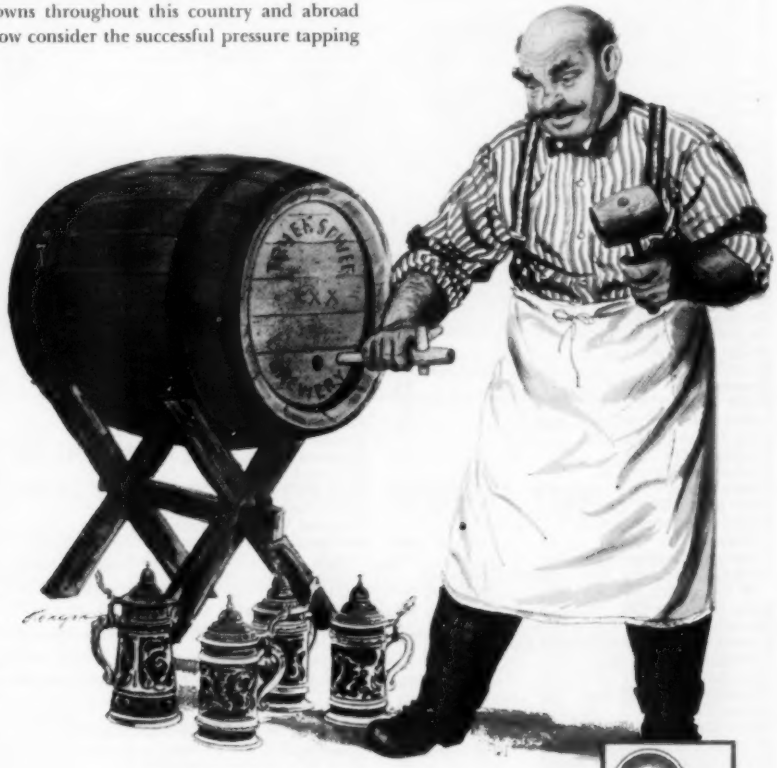

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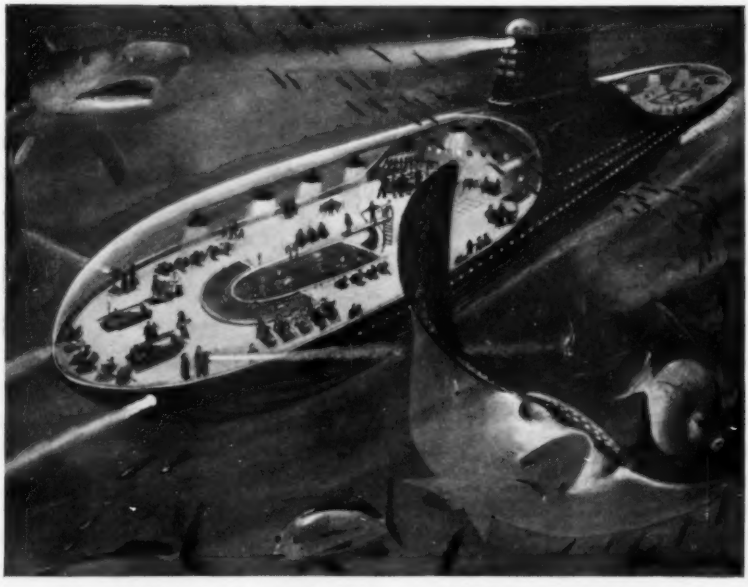

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Coming Meetings

AWWA SECTIONS

Sep. 12–14—New York Section, at Sagamore Inn, Bolton Landing, Lake George. Secretary, Kimball Blanchard, Rensselaer Valve Co., c/o Ludlow Valve Co., 11 W. 42nd St., New York 17.

Sep. 12–14—Michigan Section, Kalamazoo. Secretary, T. L. Vander Velde, Chief, Water Supply Section, Michigan Dept. of Health, Lansing 4.

Sep. 12–14—North Central Section, at Hotel Lowry, St. Paul, Minn. Secretary, Leonard N. Thompson, Gen. Mgr., Water Dept., St. Paul 2, Minn.

Sep. 17–19—Kentucky-Tennessee Section, at Hotel Patten, Chattanooga, Tenn. Secretary, J. W. Finney Jr., 553 S. Limestone St., Lexington, Ky.

Sep. 19–21—Ohio Section, at Commodore Perry Hotel, Toledo. Secretary, M. E. Druley, Dayton Power & Light Co., Wilmington.

Sep. 26–28—Wisconsin Section, at Stoddard Hotel, La Crosse. Secretary, L. A. Smith, Supt., Water & Sewerage, City Hall, Madison 3.

Sep. 30–Oct. 2—Missouri Section, at Hotel Governor, Jefferson City. Secretary, W. A. Kramer, Rm. 3, 6th Floor, State Office Bldg., Jefferson City.

Oct. 14–17—Southwest Section, at Marion Hotel, Little Rock, Ark. Secretary, Leslie A. Jackson, Mgr.-Engr., Municipal Water Works, Robinson Memorial Auditorium, Little Rock, Ark.

Oct. 18–20—New Jersey Section, at Hotel Madison, Atlantic City. Secretary, C. B. Tygert, Wallace & Tiernan Inc., Box 178, Newark 1.

Oct. 21–24—Alabama-Mississippi Section, at Battle House, Mobile, Ala. Secretary, Irving E. Anderson, Dist. Engr., Surface Water Branch, USGS, Box 2052, Jackson, Miss.

Oct. 23–26—California Section, at U. S. Grant Hotel, San Diego. Secretary, Henry J. Ongerth, Sr. San. Engr., Bureau of San. Eng., 905 Contra Costa Ave., Berkeley 7.

Oct. 24–26—Iowa Section, at Hotel Fort Des Moines, Des Moines. Secretary, J. J. Hail, Supt., Water Dept., City Hall, Dubuque.

(Continued on page 8)

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WITH THE RING-TITE COUPLING

Coming Meetings

(Continued from page 6)

Oct. 24–26—Chesapeake Section, at Southern Hotel, Baltimore, Md. Secretary, Carl J. Lauter, 6955—33rd St., N.W., Washington 15, D.C.

Oct. 31–Nov. 2—West Virginia Section, at Hotel West Virginian, Bluefield. Secretary, Hugh W. Hetzer, Engr. Gen. Office, West Virginia Water Service Co., 179 Summers St., Charleston 1.

Nov. 7–9—Virginia Section, at Chamberlin Hotel, Old Point Comfort. Secretary, J. P. Kavanagh, Dist. Mgr., Wallace & Tiernan Inc., 213 Carlton Terrace Bldg., Roanoke.

Nov. 11–14—Florida Section, at Daytona Plaza Hotel, Daytona Beach. Secretary, Jay D. Roth, City Hall, Miami Beach 39.

Nov. 12–14—North Carolina Section, at Hotel Charlotte, Charlotte. Secretary, Wilbur E. Long Jr., 1615 Bickett Blvd., Raleigh.

Nov. 26–28—Rocky Mountain Section, at Broadmoor Hotel, Colorado Springs, Colo. Secretary, Jack W. Davis, Dist. Mgr., Transite Pipe Div., Johns Manville Sales, Inc., Denver, Colo.

Nov. 29–Dec. 1—Cuban Section, at Cuban Society Engineers Bldg., Havana. Secretary, Laurence H. Daniel, Pres., Laurence H. Daniel, Inc., Baratillo 9, Havana.

OTHER ORGANIZATIONS

Jul. 30–Aug. 3—Gordon Research Conference on Ion Exchange, American Assn. for Advancement of Science, at Kimball Union Academy, Meriden, N.H. W. George Parks, Director, Dept. of Chemistry, University of Rhode Island, Kingston, R.I.

Sep. 17–21—Annual International Instrument-Automation Conference & Exhibit, New York, N.Y. For information, write Fred J. Tabery, 250 W. 57th St., New York.

Oct. 8–11—Federation of Sewage & Industrial Wastes Assns., Statler Hotel, Los Angeles, Calif.

Oct. 14–17—National Institute of Governmental Purchasing, at Conrad Hilton Hotel, Chicago. A. H. Hall, Exec. Vice-Pres., 1001 Connecticut Ave., N.W., Washington 6, D.C.

Oct. 15–17—Annual Convention, American Gas Assn., Atlantic City, N.J. For reservations, write American Gas Assn. Housing Bureau, 16 Central Pier, Atlantic City, N.J.

Oct. 15–19—American Society of Civil Engineers, Pittsburgh, Pa.

Nov. 12–16—American Public Health Assn., Atlantic City, N.J.

Nov. 27–30—National Chemical Exposition, sponsored by American Chemical Society, at Public Auditorium, Cleveland, Ohio.

1957

Feb. 3–5—Annual Midwinter Conference, Public Utility Buyers' Group, National Assn. of Purchasing Agents, at Brown Hotel, Louisville, Ky. Chairman, L. G. Wiseley, Michigan Consolidated Gas Co., 415 Clifford St., Detroit 26, Mich.

Mar. 25–29—Western Metals Congress & Exhibition, at Ambassador Hotel and Pan-Pacific Auditorium, Los Angeles, Calif. Managing Director, W. H. Eisenman, 7301 Euclid Ave., Cleveland 3, Ohio.

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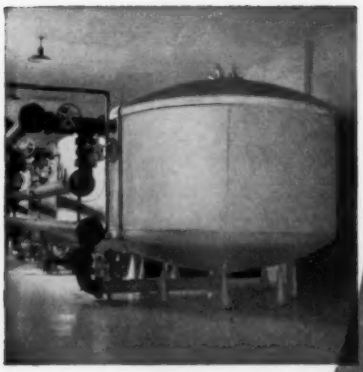
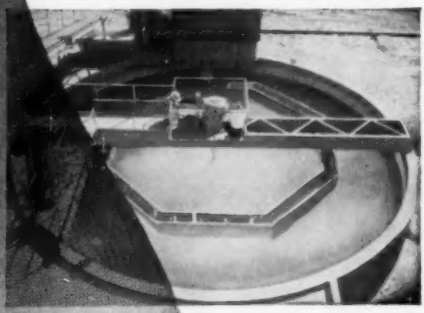
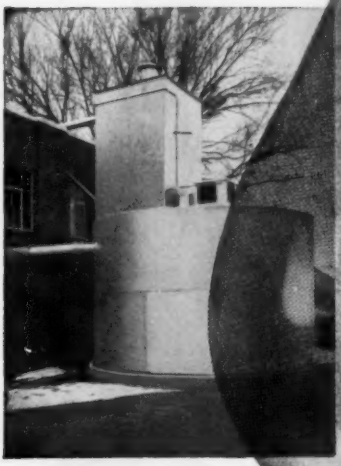


There are many reasons why municipalities and industries have installed General Filter water treating plants:





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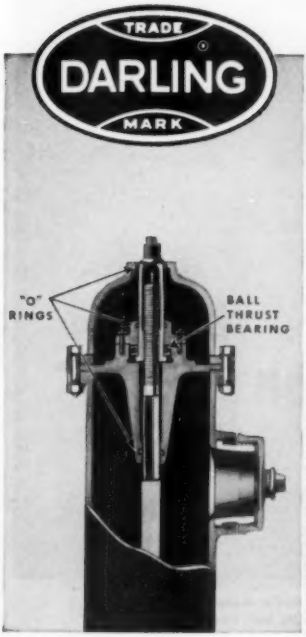

A. W. Grathwol, manager of the Sandusky, Ohio, Water Distribution Department, checking static water pressure on a newly installed Darling B-50-B hydrant.

Word is getting around about DARLING'S NEW B-50-B FIRE HYDRANT

WATER distribution men like A. W. Grathwol of the Sandusky, Ohio, Water Distribution Department, are spreading the good news about the new patented Darling B-50-B fire hydrant. It rates as the most important fire hydrant advance in 50 years.

Here are reasons why . . .

1. The "O" ring seals eliminate need for a stuffing box. Darling's B-50-B is a packless, *dry-top* hydrant that saves service time and trouble. It does away with bothersome packing gland adjustment and maintenance, assures constant lubrication of the operating threads *and prevents water from reaching the operating threads.*

2. The Darling B-50-B is a *ball-bearing* operated hydrant. Opening and closing takes less than half the usual wrench torque.

And you also get another long-time advantage of Darling hydrants . . . all inside working parts, including the main valve seat and drain valve seats, can be removed through the top *by one man* for on-the-spot inspection and maintenance. Write for Bulletin No. 5007 before you decide on any hydrant. Address . . .

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In 1925, The Board of Water Supply, city of Providence, laid a water line in Cranston, Rhode Island. Interior surfaces were hand-brushed with hot Bitumastic® Enamel.

This illustration is graphic proof of the extra value you get when you specify Bitumastic Enamel. After 28 years' continuous service, a section of this 48-inch line was removed for inspection.

Note that there is no sign of rust or tuberculation. Brush marks are still visible. The original coefficient of flow has been maintained over the years. With such performance available from Bitumastic Enamel, there is no need to specify over-sized diameter pipe, thus giving important installation as well as operational economies.

Samples of this original 28-year-old Bitumastic Enamel lining were tested. Laboratory analysis showed there was practically no change in the characteristics of the Bitumastic Enamel. This gives every reason to believe that the line will continue to give the same type of dependable performance for many more years.

Service records like this prove that it pays to use strong steel pipe, protected by Bitumastic Enamel. It's an economical combination for your large-diameter water lines. Write for complete information.

SEWAGE EQUIPMENT PROTECTION!

Koppers also makes Bitumastic® Protective Coatings for all kinds of sewage-plant equipment. Give your expensive equipment the effective protection it deserves. Write for complete data.

KOPPERS BITUMASTIC® ENAMELS

KOPPERS COMPANY, INC.

Tar Products Division, Dept. 405-T, Pittsburgh 19, Pennsylvania

DISTRICT OFFICES: BOSTON, CHICAGO, LOS ANGELES, NEW YORK, PITTSBURGH, AND WOODWARD, ALA.

"Here's why our city installed its sixth BJ Submersible"

Says L. E. Cunningham, Mayor of Belvidere, Illinois

"A dependable, protected water supply system is vital to the future growth of our residential and industrial areas."

"In the past our wells were pumped by surface motor deepwell turbines requiring pump house protection and special maintenance. In 1951 Wayne Richardson, our Waterworks Superintendent, working with R. Anderson, Engineering, replaced an old deepwell turbine with a modern Byron Jackson Submersible.

Our experience with this completely submerged pump was so satisfactory that we converted all our wells to submersible operation, having just completed the installation of the sixth unit. The combined capacity of all these units is now 5 MGD with storage facilities for 1,700,000 gallons.

The people of Belvidere are now assured a pure, low-cost water supply that is completely safe from flood or other disaster conditions."

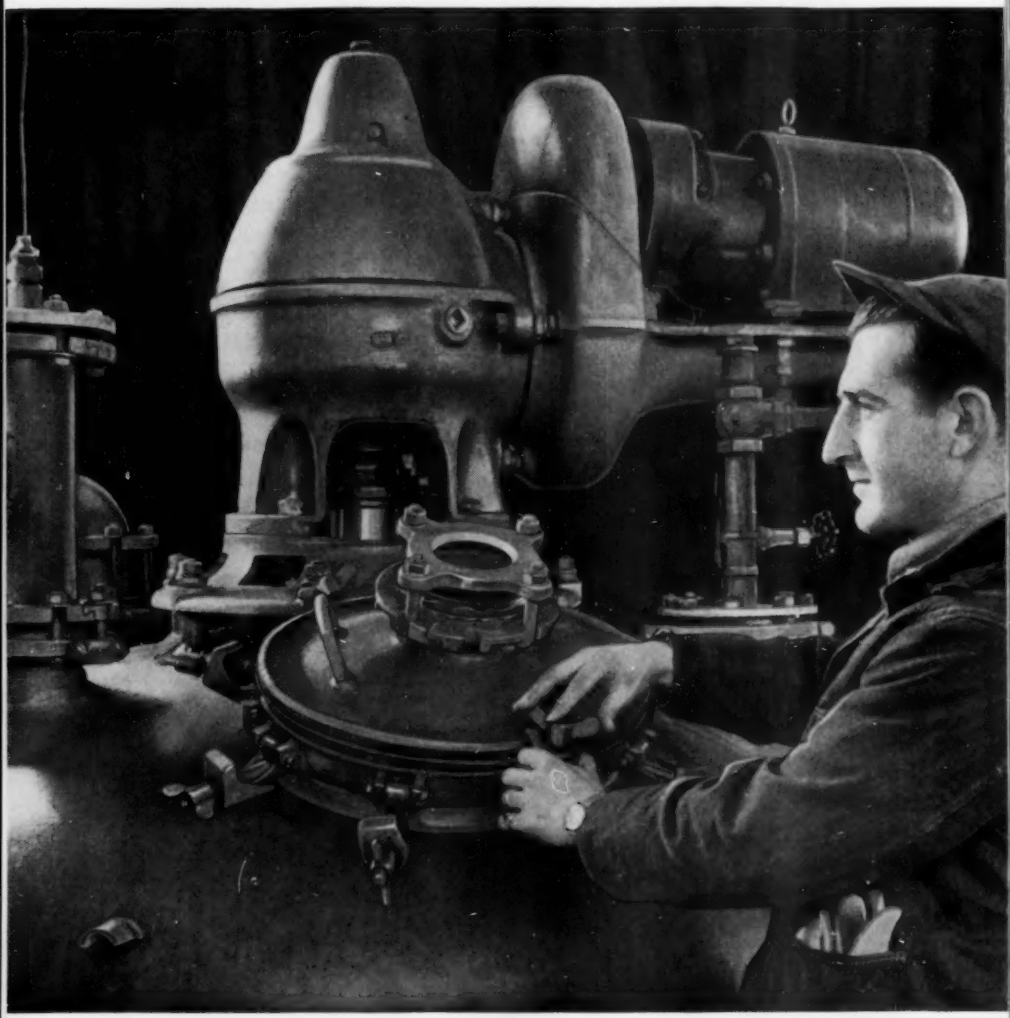
Facts about Belvidere, Illinois A friendly city of 10,000, Belvidere is located 75 miles northwest of Chicago. Its industrial advantages include abundance of water, skilled labor pool, good plant sites, low tax and utility rates.

SINCE BJ 1872

Byron Jackson

Division of Borg-Warner Corporation

P. O. BOX 2017A, TERMINAL ANNEX • LOS ANGELES 54, CALIFORNIA



Reaction kettle for making ion exchange resins at The Permutit Co., Birmingham, N. J.

Why we make our own ion exchange resins:

First—in order to offer our customers the best resins.

But the main reason we make our own resins is that we can "tailor" their molecular design, chemical and physical characteristics to best meet process and equipment requirements. It gives us greater control of capacity, efficiency, stability and mesh size.

It also supplements our production of other types of zeolites with the result that Permutit (and only Permutit) makes all widely used types of ion exchangers . . . from rugged natural greensands for simple, low-cost water softening to specialized resins for purifying chemicals.

And it enables us to offer an integrated Water Conditioning Service including water analysis, engineering, equipment, controls and the ion exchange material itself . . . from one source . . . under one responsibility. That's how "weak links" can be avoided. That's why Permutit equipment gives long years of trouble-free service.

For further information, write The Permutit Company, Dept. JA-7, 330 West 42nd St., New York 36, N. Y.

PERMUTIT®
WATER CONDITIONING
Equipment • Resins • Experience

Concrete Pressure Pipe is universally accepted for city water systems

as witnessed by its use in all major cities in the United States and Canada

Great supporting strength, high carrying capacity and long life make Concrete Pressure Pipe ideal for city water supply lines, transmission lines, and distribution mains.

Modern manufacturing plants are located so as to make Concrete Pressure Pipe available in all sections of the United States and Canada.

Member companies are equipped to manufacture and furnish concrete pressure pipe in accordance with established national specifications and standards.

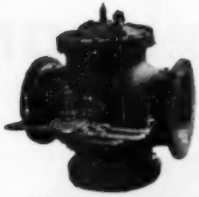
Concrete PRESSURE Pipe

WATER FOR GENERATIONS TO COME

AMERICAN CONCRETE PRESSURE PIPE ASSOCIATION
228 North LaSalle Street
Chicago 1, Illinois

1879—ROSS—1879

Automatic Valves



ALTITUDE VALVE

Controls elevation of water in tanks, basins and reservoirs

1. Single Acting
2. Double Acting

Maintains safe operating pressures for conduits, distribution and pump discharge

SURGE-RELIEF VALVE

REDUCING VALVE

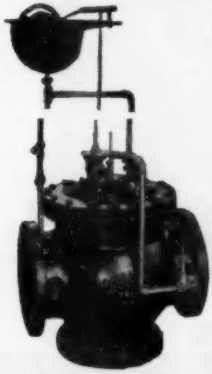
Maintains desired discharge pressure regardless of change in rate of flow

Regulates pressure in gravity and pump systems; between reservoirs and zones of different pressures, etc.

A self contained unit with three or more automatic controls

COMBINATION VALVE

Combination automatic control both directions through the valve.

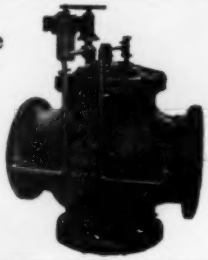


FLOAT VALVE

Maintains levels in tank, reservoir or basin

1. As direct acting
2. Pilot operated and with float traveling between two stops, for upper and lower limit of water elevation.

Electric remote control—solenoid or motor can be furnished



REMOTE CONTROL VALVE

Adapted for use as primary or secondary control on any of the hydraulically controlled or operated valves.

Packing Replacements for all Ross Valves Through Top of Valve

ROSS VALVE MFG. CO., INC., P. O. BOX 593, TROY, N. Y.

OUR FUTURE IS (and in your

MODERNIZED **cast iron**

IN THE SKIES...
hands, too)

To keep pace with America's constantly increasing demand for water, you need the help of the elements. And the public.

We can lend a hand there.

Our national advertising stresses advance planning of water facilities . . . warns millions of Americans—homeowners, apartment dwellers, industrialists about waste, pollution, the need for efficient water systems. It points out the need for new water sources, adequate treatment and distribution, realistic water rates, far-sighted water legislation . . . all the facts the public must know to cooperate intelligently with you and your fellow water officials in your difficult job of keeping America plentifully supplied with water—now and in the future.

CAST IRON PIPE. Serves for Centuries!

For over 70 public utilities in the United States and Canada, cast iron mains laid over a century ago are still serving dependably. This long, trouble-free service means lower maintenance costs . . . fewer tax dollars expended . . . fewer headaches for the waterworks officials who specify cast iron.

And today, modernized cast iron pipe, centrifugally cast, is even more rugged, uniform, durable. When needed, it's available with cement-lining to assure sustained carrying capacity throughout its generations of service.

The next time you plan extensions to your distribution system remember, cast iron pipe means the ultimate in long term reliability and economy.

Cast Iron Pipe Research Association, Thos. F. Wolfe, Managing Director, 122 So. Michigan Ave., Chicago 3, Illinois.

This rugged old cast iron water main, still serving and saving tax dollars for the citizens of *Detroit, Michigan,* is now in its 115th year of dependable service.

CAST IRON

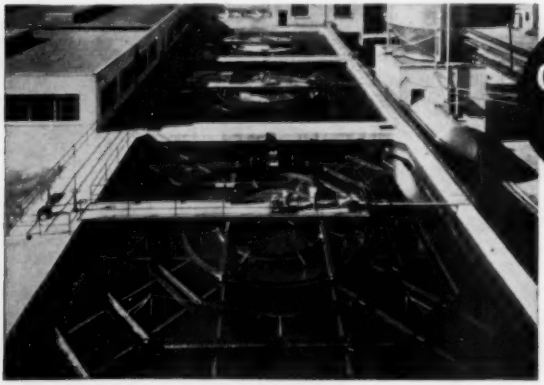
The Q-Check stenciled on pipe is the Registered Service Mark of the Cast Iron Pipe Research Association.

pipe FOR MODERN WATER WORKS

WALKER PROCESS

Clariflow

Orlando, Florida Water Treatment Plant includes three Walker Process Clariflows for lime softening as well as algae and color removal. The unit in the foreground, completed in 1954, increases the plant capacity to 24 MGD. The two original Clariflows were installed in 1949. Each unit is 56' square x 17' deep.



ORLANDO, FLORIDA

Consulting Engineers—
Robert & Co.,
Atlanta, Ga.
Gen'l. Mgr. — Orlando
Utilities Commission —
Mr. C. H. Stanton, Mgr.
Orlando Water Dept.—
Mr. L. L. Garrett

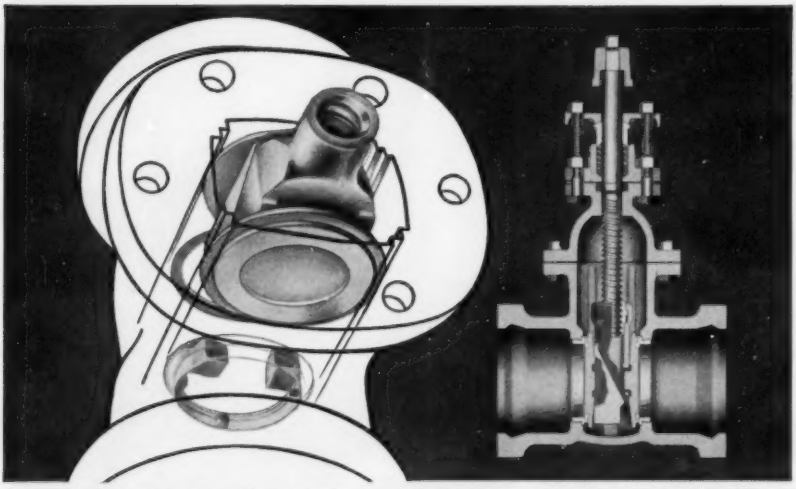
The Clariflow combines flocculation, good fluid mechanics and clarification in a relatively small tank. Mixing, flocculation, stilling and sedimentation are independently operated and controlled. The positive control of flocculation and clarification enables the operator to readily select the most economical method of operation when handling changeable water conditions.

Short circuiting tendencies are eliminated by means of exclusive multiple, tangential diffusers which simultaneously and equally distribute the flow. Balanced multiple surface weir troughs make efficient use of short detention periods and insure clarified overflows.

The Clariflow is applicable wherever there is a municipal or industrial need for water or waste treatment. It can be used in all operations including combined intimate chemical homogenizing, flocculation and clarification in rectangular, square or circular basins. The Clariflow gives excellent results in the treatment of municipal and industrial water for—softening—turbidity removal—color removal—algae removal. Industrially it is universally used in—oil separation and emulsion breaking plants—blast furnace flue dust thickening—paper stock reclamation.

Write for bulletin 6W 46.

WALKER PROCESS EQUIPMENT INC.
FACTORY—ENGINEERING OFFICES—LABORATORY
AURORA, ILLINOIS



The outline drawing shows how the simple, 4-piece disc assembly is held in alignment by the guide ribs. As the disc assembly descends on the stem, the lower spreader strikes a boss in the body. Further closing movement exerts pressure on the spreaders in a wedging action, forcing the parallel disc tightly against the seats.

New Crane disc assembly and guide rib design improve efficiency of AWWA valves

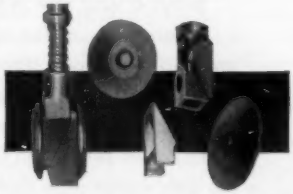
A simple, 4-piece disc assembly and a unique guide rib design make Crane's new AWWA double-disc gate valves a wise choice for mains and distribution lines.

The double-disc assembly, held by the new guide rib design, cannot become disengaged in service. The trunnion-mounted discs are free to rotate, thus preventing concentrated wear on both discs and seats.

Crane double-disc gate valves meet all AWWA specifications—and more! For example, the 2-piece gland and gland flange with ball type joint which prevents stem binding despite uneven pull-up on gland bolts.

LITERATURE AVAILABLE

These Crane quality AWWA valves are available in sizes from 2 to 12 in. Circular giving complete specifications available from your local Crane Representative, or by writing to address below.

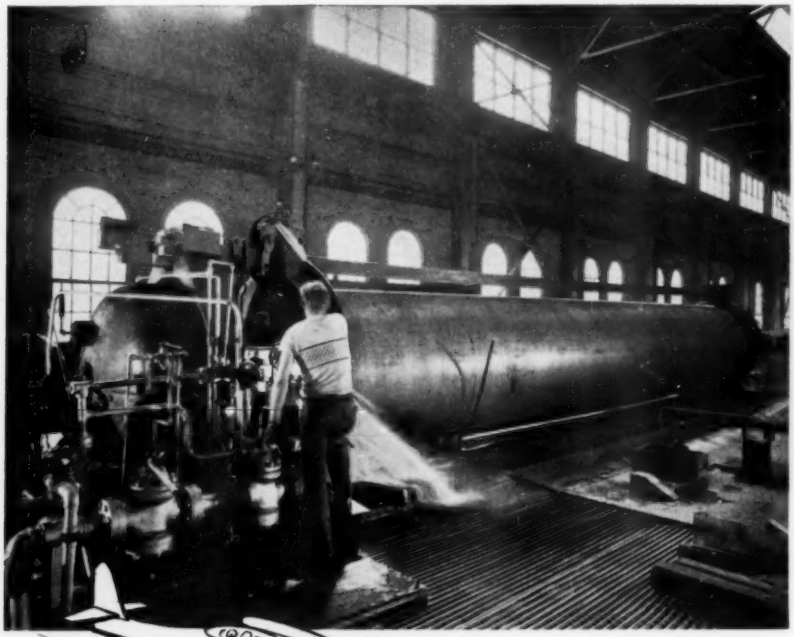


This view of disc assembly shows simplicity. Assembly is suspended from stem, which engages upper spreader. Discs are suspended from upper spreader on trunnions.

CRANE VALVES & FITTINGS

PIPE • KITCHENS • PLUMBING • HEATING

Since 1855—Crane Co., General Offices: Chicago 5—Branches and Wholesalers in All Areas



STEEL PIPE
HAS ITS TEST PILOTS, TOO!

Like the more glamorous aviation test pilot, the steel pipe tester plays a vital role in the story of safety. His job demands the greatest caution and concern to assure your water pipe meeting the highest standards of safety, strength and performance.

This specialist hydrostatically tests *every* length of steel pipe to at least twice its required working pressure . . . to guarantee you maximum protection against breakage and leakage. And steel pipe is the *only* water pipe that undergoes this rigid individual testing process.

When you need a pipe that you know in advance will pass all the tests of stress and time—you're smart to specify STEEL pipe.

"WHEREVER WATER FLOWS—STEEL PIPES IT BEST"

STEEL PLATE FABRICATORS ASSOCIATION

79 W. MONROE ST., CHICAGO 3, ILL.

CITY OF BEREA, OHIO *cuts costs*

New Cochrane SOLIDS-CONTACT REACTOR *combines*

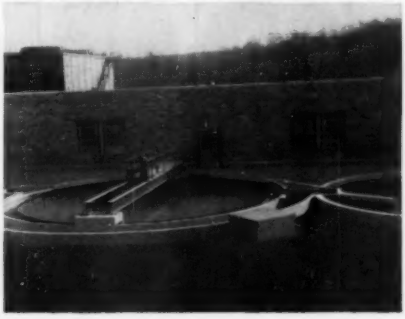
mixing, precipitation, sludge concentration, clarification & softening

The relatively hard, turbid Rocky River supply for the city of Berea, Ohio, is quickly reduced in hardness to approximately 3.5 gr/gal and to a turbidity of less than 10 ppm by the Cochrane Solids-Contact Reactor shown above. An existing concrete basin 28′ square x 15′ deep was modernized by conversion to the Solids-Contact type. Using hydrated lime, soda ash and alum, the Reactor treats over 2,000,000 gpd at surprisingly low cost.

Because the design of the Cochrane Reactor provides higher quality treated water faster, in less space, with minimum chemicals, their use has grown tremendously for municipal applications. High slurry strength in the reaction zone speeds precipitation—there is very little waste water. Automatic desludging saves time and labor. In addition to softening and clarification, Cochrane Reactors remove color, taste, odor; reduce alkalinity, silica, fluorides, etc.

Cochrane's background in water conditioning makes possible the installation of complete systems under a single responsibility for continued, consistent performance. Write for Publication 5001-A and case history reprints.

Cochrane also manufactures circular reactors



Cochrane

CORPORATION

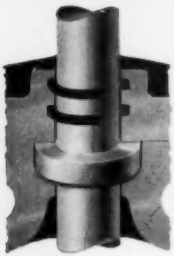
3124 N. 17TH STREET, PHILADELPHIA 32, PA.

NEW YORK • PHILADELPHIA • CHICAGO

Representatives in 30 principal cities in U.S.; Toronto, Canada; Paris, France; Mexico City, Mexico; La Spezia, Italy; Havana, Cuba; Caracas, Venezuela; San Juan, Puerto Rico; Honolulu, Hawaii; Malmo, Sweden; Santiago, Chile. **Pottstown Metal Products Division**—Custom built carbon steel and alloy products.

Demineralizers • Hot Process Softeners • Hot Zeolite Softeners • Dealkalizers • Reactors
Deaerators • Continuous Blowoff • Condensate Return Systems • Specialties

KENNEDY A.W.W.A. Gate Valves
Give You More Value For Your Valve Dollar . . .



O-RING SEALS

Provide excellent seal, prevent binding of stem and result in very easy valve operation. KENNEDY O-Rings are located above collar, permit repacking under pressure. Optional on KENNEDY non-rising stem A.W.W.A. valves.

STRONGER CONSTRUCTION

Rugged design and construction features iron that is 50% stronger than ordinary cast iron to keep KENNEDY Valves doing a better job over a longer, more dependable operating life.

KENNEDY Fig. 56
A.W.W.A. Standard Iron-Body Double-Disc Gate Valve, with Bell Ends.

KENNEDY manufactures a complete line of water works valves. Sizes range from 2" to 48" including non-rising stem and outside-screw-and-yoke valves. These valves feature the hook-and-wedge type disc mechanism for easier, better and more positive valve operation. A wide variety of pipe connections are available.

For maximum service life, for the greatest value for your valve dollar, specify KENNEDY valves and fire hydrants. Remember, KENNEDY means dependability in valves, fire hydrants and accessories.

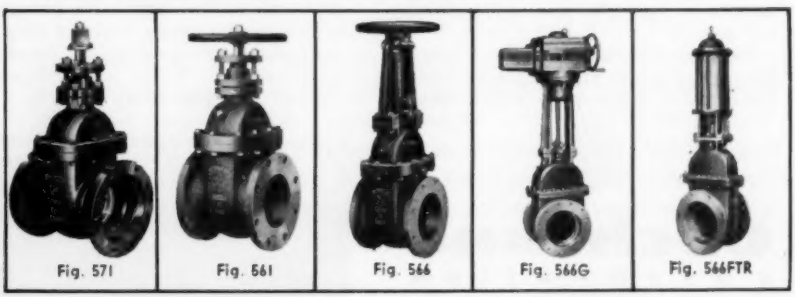


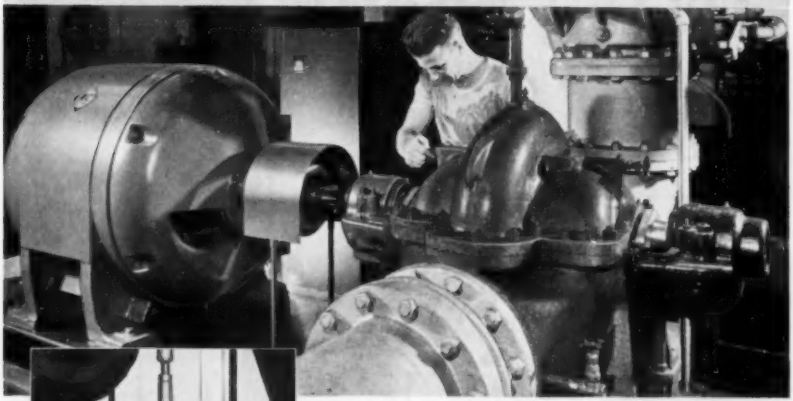
Fig. 571 | Fig. 561 | Fig. 566 | Fig. 566G | Fig. 566FTR

Write today for NEW A.W.W.A. Valve Bulletin

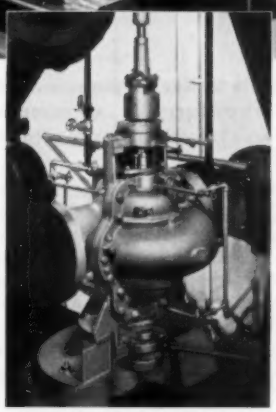
THE KENNEDY VALVE MFG. CO. • ELMIRA, N. Y.
VALVES • PIPE FITTINGS • FIRE HYDRANTS

OFFICE AND WAREHOUSES IN NEW YORK CHICAGO SAN FRANCISCO ATLANTA SALES REPRESENTATIVES IN PRINCIPAL CITIES

Allis-Chalmers PUMPS Solve a Water Works Problem



Newly installed Allis-Chalmers horizontal high-lift, 5215-gpm, 220-ft head pump, driven by 350-hp, 1170-rpm Allis-Chalmers motor, replaces old Allis-Chalmers pump-motor combination.



This new Allis-Chalmers vertical pump, rated 5215 gpm at 55-ft head and driven by a 100-hp, 1170-rpm induction motor, pumps raw water out of Lake Michigan into the filter beds.

Waukegan Increases Capacity, Modernizes Plant . . . Again Specifies Allis-Chalmers

Although pumps, motors, control and switchgear, installed in 1928, were still giving dependable, economical service, need for increased capacity indicated replacement of some equipment at the Waukegan (Ill.) Water Works.

The first step in modernizing the plant was to replace two of the original 16 pumping units, one vertical and one horizontal. This included new Allis-Chalmers pumps with increased capacity, new motors, and new control.

You get MORE than a Pump . . . When You Specify Allis-Chalmers

You can take advantage of Allis-Chalmers wide experience in supplying pumps to all industries. You are assured of modern design, heavy-duty construction and correct application aid — all resulting in added years of dependable, economical, day-after-day pumping service.

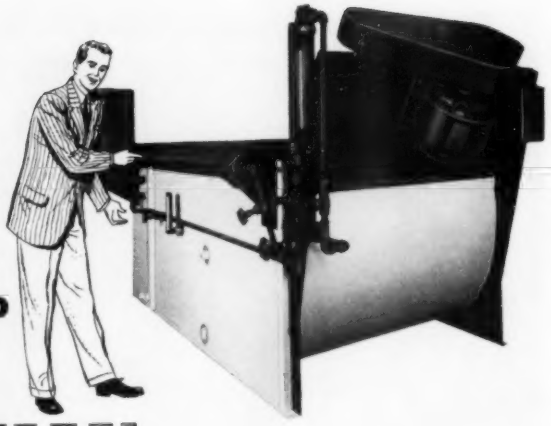
Allis-Chalmers is the only company that can offer you "One-Source" responsibility, with a complete unit — pump, motor and control — all built to work together. For MORE information, call your local A-C office, or write Allis-Chalmers, General Products Division, Milwaukee 1, Wisconsin.

A-5133

A-C ALLIS-CHALMERS

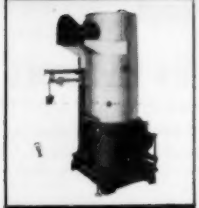
ALLIS-CHALMERS

it's OMEGA for MECHANIZED SLAKING



OMEGA BELT GRAVIMETRIC FEEDER

Weighs and feeds lime on short conveyor belt. Feeding rates from 10 to 10,000 lbs. per hour. Accuracy within ±1%.



OMEGA UNIVERSAL VOLUMETRIC FEEDER

Dependable feeding—without clogging. Feeding ranges from 1 lb. to 5,000 lbs. per hour. Accuracy within ±5% by volume.

OMEGA LOSS-IN-WEIGHT GRAVIMETRIC FEEDER

Unique principle insures full control and conservation of lime, plus wide adjustability. Rates from ½ to 5,000 lbs. per hour. Accurate within ±.5%.

OMEGA Lime Slaker produces smooth, clinker-free slurry . . . eliminates shut-downs for grit and clinker removal!!

For thorough slaking that makes the most of every ton of lime you buy, investigate the Omega Lime Slaker. Adaptable for use with either Gravimetric or Volumetric Feeders, this unit produces a completely slaked, creamy slurry. Check these built-in advantages:

- **High speed propeller-type mixers** break up hard particles for thorough slaking.
- **Insulated body** and built-in heat exchanger assure peak slaking efficiency.
- **Thermostatic control** maintains high slaking temperatures, yet prevents overheating.
- **Vapor removal device** of adequate capacity protects feeder from hot, dust-laden vapors.
- **Like all B-I-F products,** OMEGA Lime Slakers consistently prove why "the best costs less". OMEGA'S complete line enables you to standardize on one dependable source of chemical feeders. Saves nuisance and expense of divided service and responsibility.

Send for Bulletin 40-E1B to **OMEGA MACHINE COMPANY,** 365 Harris Avenue, Providence 1, R. I. . . . division of

B-I-F INDUSTRIES
PROVIDENCE, RHODE ISLAND
METERS FEEDERS CONTROLS

BUILDERS-PROVIDENCE, INC. • PROPORTIONEERS, INC. • OMEGA MACHINE CO.

USE NORTHERN GRAVEL
for
RAPID SAND FILTER

FILTER SAND SPECIFICATIONS are carefully laid out. The Effective Sizes and Uniformity Coefficients used by Consulting Engineers and also recommended by the American Water Works Association are the result of long years of research and experience.

The Northern Gravel Company is equipped to give you prompt shipment whether it be one bag or many carloads, exact to specification. Filter sand can be furnished with any effective size between .35 MM and 1.20 MM.

CHEMICAL QUALITY of the filter sand is also important. It must be hard, not smooth and free of soluble particles. This requires perfect washing, and grading facilities. We have every modern device for washing, drying, screening and testing.

FILTER GRAVEL supporting the Filter Sand Bed must be, in turn, properly graded to sizes calculated to support the Filter Sand, and be relatively hard, round and resistant to solution.

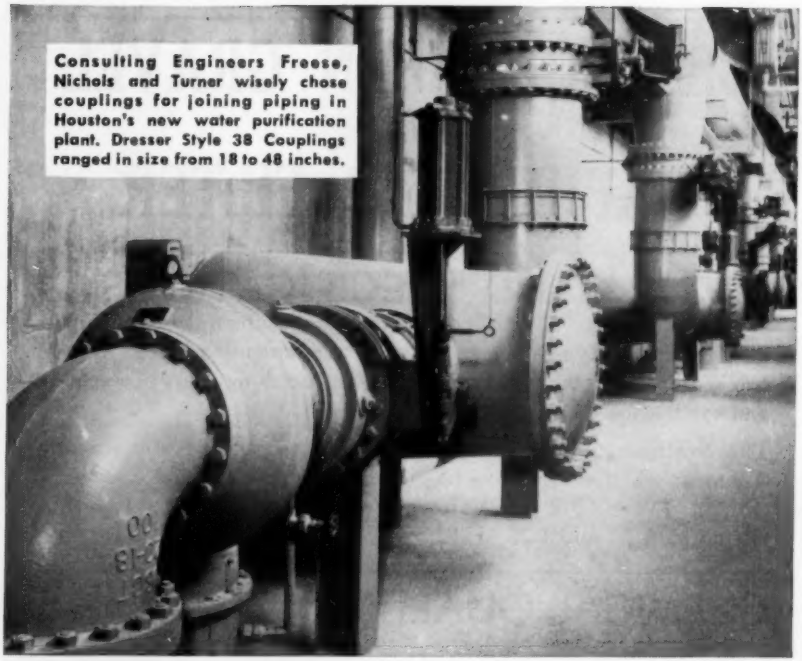
The new Northeast Station in the City of Detroit, which is scheduled for completion in 1956, is one of the major projects included in the water department's expansion program. The Northern Gravel Company furnished 120 carloads of filtering materials for the 48 rapid sand filters incorporated in this plant.

Northern Gravel has no equal in facilities and our reserves of both sand and gravel are inexhaustible. Northern Gravel Company has been in business over 40 years. We guarantee uniformity of products and our records enable us to duplicate your requirements on short notice. Our location is central and we have commodity rates in every direction.

NORTHERN GRAVEL COMPANY

Muscatine, Iowa

P.O. Box 307 **Phone: Amherst 3-2711**



Consulting Engineers Freese, Nichols and Turner wisely chose couplings for joining piping in Houston's new water purification plant. Dresser Style 38 Couplings ranged in size from 18 to 48 inches.

Houston solves piping problems for good

Steel pipe and Dresser Couplings provide flexible installation, easy accessibility

Houston's new water purification plant, a part of the San Jacinto River Water Project, integrates some of the latest developments in industrial design with time-proved principles of municipal water treatment.

The new plant reflects the wise use of steel pipe and mechanical couplings where weight, price and greater flexibility are important factors.

Consulting Engineers Freese, Nichols and Turner of Houston specified Dresser Couplings teamed with steel pipe "to provide for expansion and contraction of long runs of pipe, to provide for flexibility in the installation and possible future dismantling of large pipe assemblies, and to connect steel pipe to concrete hydraulic structures."

Whenever you join pipe—especially near valves or any type of mechanical installation—provide easier installation and permanence with safe, sure Dresser Couplings.

Regardless of vibration or expansion-contraction, the resilient gaskets in Dresser Couplings hold bottle-tight for life. This built-in flexibility, particularly important between rigid flanges, serves to prevent piping strain from reaching valve casings and working parts. Result: greater efficiency, longer life.

Installation with Dresser Couplings requires only a wrench, is fast and simple. You can forget time-consuming procedures because Dresser Couplings compensate for slight misalignments, facilitate installation of set, rigid equipment on piping. These couplings also permit easy entrance into lines for maintenance or repair of equipment.

No other method of pipe joining provides all these proved advantages. Make sure you have foolproof, leakproof Dresser Couplings on your next water, sewerage or industrial project. For complete information, call your local piping supply house or write Dresser Manufacturing Division, Bradford, Pa. Sales offices in: New York, Philadelphia, Chicago, S. San Francisco, Houston, Denver, Toronto and Calgary.

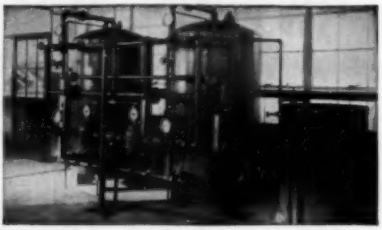
ONE OF THE DRESSER INDUSTRIES

Roberts Filter *means...*

MUNICIPAL WATER PURIFICATION

The combined capacity of Roberts-equipped filtration plants is well over 5 billion gallons (5,000,000,000) per day. Regardless of the size of the plant or the nature of the filtration problem, Roberts Filter can be depended upon for equipment that is reliable in years of service.

INDUSTRIAL WATER RECTIFICATION

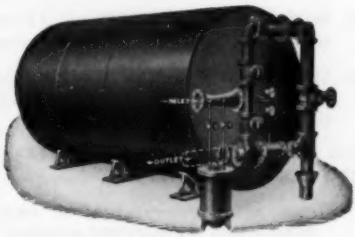


Water treatment has long been a specialty of Roberts Filter. Zeolite water softeners are guaranteed to meet all requirements for which recommended, and are available in a wide range of capacities. Roberts water conditioning equipment is widely used to control precisely the desired chemical content of water for industrial use.

SWIMMING POOL RECIRCULATING SYSTEMS

The combination of thoroughly clarified water and efficient recirculation are features for which Roberts pools are famous. Systems for both outdoor and indoor pools are designed and installed by men long experienced in the conditions peculiar to a successful swimming pool installation.

PRESSURE FILTERS



Closed pressure filters have wide usage where gravity filters are not justified. Roberts vertical filters are available in standard types from 12" to 96" diameter; horizontal pressure filters are all 8'0" in diameter and in varying lengths from 10'0" to 25'0".

When you think of good water—think of Roberts Filter

MECHANICAL EQUIPMENT BY ROBERTS FILTER MFG. CO.

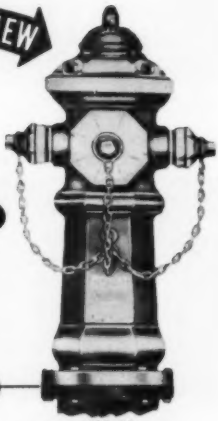
Roberts Filter
Manufacturing Company • Darby, Penna.

A. W. W. A.

HYDRANTS

EASY TO OPERATE AND MAINTAIN

NEW

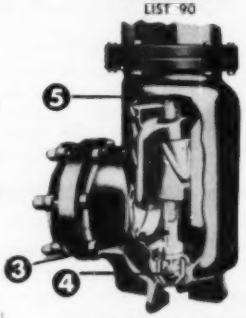


LIST 90

Ludlow hydrants incorporate all the advantages and meet all the requirements for safety, durability and economy.

HERE'S WHY:

1. **POP-OFF SLEEVE COUPLING:** releases hydrant head from stem in event of traffic damage.
2. **BREAKABLE GROUND LINE FLANGE** eliminates digging.
3. **NO FLOODING.** Closed hydrant remains locked—even in event of traffic accident.
4. **NO FREEZING.** Positive drip action, at extreme bottom, assures complete drainage.
5. **90,000 TENSILE STRENGTH** rolled Everdur threaded stem section completely eliminates stem failure.
6. **LESS OPERATING TORQUE.** New, non-binding upper and lower "O" rings trap lubricant . . . assure positive water tight seal . . . require no servicing.



MECHANICAL JOINT END

THE RESULT: LOW INITIAL COST—YEAR AFTER YEAR—OF TROUBLE-FREE SERVICE.

"NO POSSIBLE DIFFERENCE IN FIRST COST CAN OVERBALANCE THE PERPETUAL ECONOMY OF QUALITY."

15

Ask for Bulletin 54-H

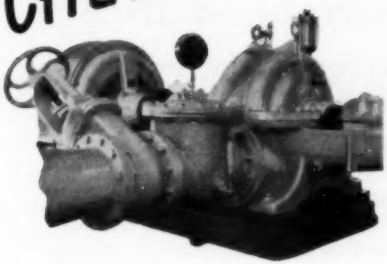
THE LUDLOW VALVE MFG. CO., INC. TROY, N.Y.

TRADE LUDLOW MARK

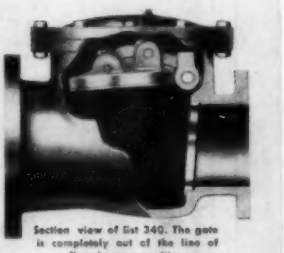
GATE VALVES • FIRE HYDRANTS • SQUARE BOTTOM VALVES
CHECK VALVES • TAPPING SLEEVES • AIR RELEASE VALVES

CLEARWAY—QUIET CLOSING

CHECK VALVES



List 340 Clearway, quiet closing check valve: increasing type with flange ends.



Section view of list 340. The gate is completely out of the line of flow in open position.

Low cost insurance against severe slam on pump shut down is provided in the Rensselaer clearway, quiet closing, non-slam check valve.

In operation, the gate is normally completely out of the line of flow and head losses are extremely low, even at high velocities. The lever arm and adjustable spring force the gate toward its seat on pump shutdown and the gate seats firmly at the instant of zero velocity, without slamming.

The List 340 Spring Check Valve may be installed in either horizontal or vertical pipe lines and is made in sizes up to 30 inch and pressures of 200 lbs. with flange ends only. Dual levers and springs may be provided for extreme conditions. All parts are readily accessible through the large removable cover.

The increasing type illustrated bolts directly to the pump discharge flange, eliminates increasers and saves valuable space. For locations where slamming is not anticipated, the Rensselaer list 37 without lever and springs is recommended. For higher pressures, the List 39 operates on non-shock cold water, oil or gas up to 400 lbs. W.P.

Rensselaer VALVE CO.

DIVISION OF THE LUDLOW VALVE MANUFACTURING CO., INC.

Ask for Bulletin "E"

TROY, NEW YORK

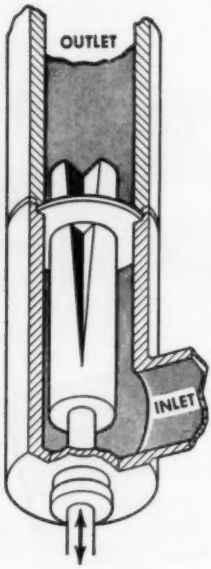
GATE VALVES • FIRE HYDRANTS • SQUARE BOTTOM VALVES
CHECK VALVES • TAPPING SLEEVES • AIR RELEASE VALVES

THE WALLACE & TIERNAN

V-NOTCH

variable-orifice

... a new concept for chlorinator control



The W&T V-notch Variable-Orifice is the heart of a new chlorinator control system. With the system, a constant differential vacuum is maintained across the V-notch Variable-Orifice and the flow of chlorine gas is adjusted by varying the area of the orifice.

The W&T V-notch Variable-Orifice has these features:

- Wide range adjustment of chlorine flow
- Excellent reproducibility of flow by a simple adjustment
- Same % accuracy over entire wide flow range

As can be seen in the diagram, the V-notch Variable-Orifice is formed by a cylindrical plug positioned inside a matched, circular ring. A V-shaped groove is machined along the length of the plug. The groove starts near one end of the plug and gradually enlarges to its maximum width and depth at the other end of the plug. The V-notch Orifice is formed at the intersection of the grooved plug and the matched ring. The orifice area is varied simply and positively by positioning the plug backward or forward inside the fixed ring.

The V-notch Variable-Orifice is just one design feature of the new W&T V-notch line of Chlorinators. Get all the facts about the W&T V-notch chlorinators before you buy chlorination equipment.

WALLACE & TIERNAN INCORPORATED

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Journal

AMERICAN WATER WORKS ASSOCIATION

VOL. 48 • JULY 1956 • NO. 7

Public Water Supply and the Future

Paul Weir

The address of the incoming President of the American Water Works Association, presented on May 10, 1956, at the Diamond Jubilee Conference, St. Louis, Mo., by Paul Weir, Gen. Mgr., Water Works, Atlanta, Ga.

WATER, or the quest for water, has dominated human life throughout history. The water works of the future depends on *men*—the scientist, the administrator, the citizen-customer—more than on *trends of the time*. Immediate and long-range plans for progress make it necessary that these three groups work together very closely. The breath-taking increase in population and the constantly rising standard of living are the two main factors of the demand for better and more efficient products and services from all types of businesses and utilities.

The nation doubled its population during the first half of the present century, and then added an equivalent to the entire population of Canada during the last 5 years. The President's Materials Policy Commission predicts that there will be 49,000,000 more people in the United States by 1975, making a total population of 221,000,000. That means a population increase of almost one-third.

Our combined water consumption for household, industrial, and agricultural purposes exceeds 1,000 gpd per person. We use, for example, 40,000 gal of water to process 1 ton of wood pulp and 60,000 gal for a ton of steel. Therefore, when you purchase an automobile you indirectly buy about 400 tons of water. (The automobile industry built about 10,000,000 cars and trucks last year.)

Since the water works industry is a basic utility and a vital factor in our economy, it will, of necessity, be one of the pioneers and a pace-setter of progress. But as Oliver Wendell Holmes said, "It is not where we are that counts, but where we are headed."

The water supply industry has literally pulled itself up by its bootstraps during the past 75 years. It is estimated that the present replacement value of public water utilities would exceed 23 billion dollars. It is estimated also that we will have to spend more than $300,000,000 annually to

keep up with the ever-increasing demands for water service.

If the predicted rate of population growth continues until 1975, the average municipal utility will be called upon to furnish about 50 per cent more water than at present. Today, the municipal water supply furnishes about 150 gpd per capita and by 1975 it is anticipated that 225 gpd per capita will be required. This means that we will be confronted with about a 3 per cent increase in water consumption each year.

This year, it is our aim to focus attention on "Plans for the Future." All progress depends upon our ability to cope with present demands and adequately plan for future expansion. Indeed, "silent service is not enough" if we are to attain our rightful place in progressive industry and the national economy.

The Scientist

The scientist in his research laboratory spearheads the constant search for newer and better ways of doing things. He, and the results of his research, will be responsible for new building materials, wonder drugs, miracle fabrics, plastics, chemicals, steel, metal and nonferrous products, in addition to atomic and electronic devices that will challenge man's imagination.

Improved techniques will make it possible for the consulting engineer to incorporate these new materials and designs in his plans for future water works plants and systems.

Big business firms like General Motors, Westinghouse, and many others have research departments and are spending millions of dollars experimenting with and perfecting new methods and products. General Electric is constantly reminding us that "progress is their most important product," and by the same token, progress is the water industry's greatest challenge.

Now is the time for all business, large and small, to sponsor and encourage the youth of our nation to choose scientific and engineering careers as vocations. Adequate scholarships, opportunities for advancement, and higher salaries will be the determining factors in enlisting our future scientists and engineers. The Association is doing this in a very admirable way with the annual Harry E. Jordan Scholarship Award, which was established by the Water and Sewage Works Manufacturers Association in 1954. The water works industry *must* keep pace with modern business and also entice trained young people into our field. In order to do this, a general lifting of sights is necessary as we build for the future.

Several years ago, AWWA launched a drive to alert officials to the need for future planning. The theme of this great undertaking was "Blueprint *NOW*." This program inspired hundreds of systems to take stock of their existing facilities and personnel, in order that plans might be developed to catch up with war-deferred maintenance and meet the ever-increasing demands for service.

Scientists and consulting engineers can create and blueprint, but they are dependent upon management for the execution of their plans.

The Administrator

The administrator is the key executive in a water works system. He should be surrounded with technical and progressive associates. He should

have a technical background, with a general knowledge of the needs of the entire system, and the ability to coordinate and integrate all phases of the department's operation.

His chief concerns should be:

1. To furnish the entire community with an abundant and dependable supply of potable water at a reasonable price
2. To enlist adequately trained personnel for each specific job, with commensurate salary
3. To keep the mayor and other city officials constantly informed regarding the wisdom of securing sufficient funds to finance a conservative program of progress
4. To coordinate and relate scientific findings to present-day operation and future advancement of the department
5. To explore every avenue of approach to educate the citizen-customer on the value of a dependable water supply system through contacts with manufacturers, newspapers, radio, television, civic groups, schools, colleges, and other organizations. Willing Water, our public relations goodwill ambassador, has been most helpful in this program. Annual reports and departmental bulletins, too, are valuable channels of information for employee and citizen alike.

The American Water Works Association is a most vital factor in helping the administrator integrate scientific research into everyday operating practices. The four technical divisions of AWWA include Management, Resources, Distribution, and Water Purification. The purpose of each division is to serve the members in the particular field and provide them with authenticated data and material. Supervisory personnel do a better job when they are informed members of the AWWA. It is management's responsibility to encourage every water works employee to become a member of our great Association. We can, for instance, tell nonmembers of the following benefits of AWWA:

1. Each member receives the JOURNAL monthly. This valuable publication includes interesting articles on every phase of water works operation. It will inspire each employee to do a better job.
2. The membership offers opportunities to attend sectional and national conventions, to participate on programs, to share mutual problems and their solutions, and to see firsthand displays of the latest in materials and equipment of leading manufacturers.
3. The cost of membership is only $10 per year.

I want to challenge each water works member and manufacturer to make a thorough study of the employees under his direction in regard to their affiliation with AWWA. In unity there is strength and progress—the American Water Works Association's progress! Let's have 12,000 members by 1957!

The administrator has the responsibility of providing incentive for his employees. He must be constantly alert to working conditions, which should be examined regularly and improved as need arises. Monthly employees' conferences are recommended as morale builders; pride in one's job and loyalty to the entire department are basic factors in satisfied employees. Cooperation and recognition often develop leadership among the ranks. Salaries will have to be adjusted up-

ward in order to keep the trained employee in the water works field. Every system should operate its own "in-service training" program.

There is a little story we tell our folks down South about sweet potatoes, which the Georgia farmers call yams or "sweet taters." In choosing employees, we tell them of the many other kinds of "taters," such as, the "Agi-tater," who is constantly "negative" and is always finding something to grumble about. The "Commentater," is the talker who always knows "all the answers" but seldom puts them into practice. Next, there is the "Spec-tater": he just wants to sit on the sidelines of the departmental activities and let the others do the committee and interdepartmental work. There is another "tater" who is the "Dic-tater," and he wants to have *his* way in everything; if he doesn't run things he is unhappy.

All of these "taters" are the undesirable ones, but the "tater" we want in our water department is the "sweet tater" who cooperates, takes his turn in all activities, promotes ways and means that will help the department, and creates "goodwill" among his coworkers and the public he serves.

Our aim is to encourage employees to be "sweet taters" and, regardless of what part of the country an administrator comes from, we hope he looks for and chooses "sweet taters" as his employees.

As the administrator looks into the future through windows of opportunity, progress, and service, he should be alert to the latest developments in his field. For instance, he should have a passing knowledge of electronic accounting equipment (sometimes called "mechanical brains"), microfilming records, latest methods of water purification, plastic equipment and service lines, automation in the effective operation of production plants, centralized meter reading, improved construction equipment, atomic power plants and equipment, potential use of solar energy, latest methods of communication, and improved transportation. He must have the ability to adapt these modern devices to his present system and relate them to future planning.

An administrator should also have an interest in corrosion control, rainmaking methods, comfort conditioning in the home, office, and factory, and ultrasonic inspection of water mains for defects.

Plans for the future of the water works industry will require more adequate financing than at present. This may require higher water rates, which, at the present 5¢ per ton, are much too low in comparison with other essentials of life, such as bread at $200 per ton, milk at $250 per ton, and meat at $1,500 a ton.

The major sources of revenue for water works systems include: general taxation, special assessments, and water sales. For major capital improvements and expansion, however, general obligation or revenue bonds are usually issued. These are retired from current revenue. All revenue derived from the sale of water should be used exclusively for departmental operation and not allocated to other uses. This is a must if we are to maintain our proper place of prominence as one of the Big Four utilities.

Other revenue-producing measures may be devised in the future; at present, many water departments are called upon to collect sewer rental charges based upon a percentage of water used,

and this will probably become general practice in the next few years. The mere presence of an adequate water supply in a community is a value to taxable property which should share a reasonable part of the cost.

The Association must continuously alert its officials to the requirements of an ever-expanding utility and then educate them to the fact that the cost of modern plants and efficient personnel is an investment that pays dividends. An efficiently run department that provides adequate service for the citizenry of a great city or thriving community is a great asset.

The Citizen-Customer

The citizen-customer is the taxpayer who foots the bill for services rendered. He may be a homeowner, a shopkeeper, or an industrialist, but his wants, needs, and willingness to pay determine the ultimate progress that is made in any municipality. The water works utility may have been a prime factor in his decision to locate in any particular city, and once he becomes a customer it is the responsibility of the water department to provide adequate service and fire protection at a reasonable cost.

National surveys show that we have shifted from a rural to an urban economy. Now the urban population is moving out into the suburbs because of the availability of water service and of other utilities, and improved transportation. Business is being decentralized also. Industrial areas are springing up in the fringe zones where public utilities, parking, and railroad sidings are available. All of this wide expansion places a greater responsibility to serve the most important person in the world—the citizen-customer, wherever he may choose to live, work, or play.

To give better service to our customers in the future, it is necessary to conserve our natural resources today. Atomic power for domestic and industrial purposes is a magic and revolutionary part of the future; water conservation through dams and reservoirs will play a vital role in the development of that new and wonderful giant. Other factors in conserving our natural resources with dams are flood and drought control. When rivers run rampant, life is lost and millions of dollars worth of damage is done to homes, crops, and business. All this loss can be changed to gain if intelligent planning and cooperative effort is used to control our nation's waterways. Protracted droughts, on the other hand, have wrought equal destruction to our nation. During the past few years every section of our country has lost millions of dollars because of these two destructive forces.

At present, one of the biggest potential problems with surface water supplies is stream pollution. Public opinion and adequate laws carry great weight in its correction or prevention. Not to use our streams to the limit permitted by realistic safeguards to life and health would be wasteful, however. To allow them to become waste carriers or open sewers, on the other hand, would be tragic. There is a direct relation between water pollution and water scarcity. Pollution abatement is one of our most effective water conservation measures.

We are fortunate that the total water resources of the nation have changed little during the past 75 years. The average national rainfall is about 30 in. annually. Of this amount, about 11 in.

is used by vegetation, 10 in. evaporates from ground and water surfaces, and another 8.5 in. runs off the surface of the land or through the ground directly into streams and thence into the oceans. Only about 1 in. of all this water (less than 5 per cent of total rainfall) is intercepted annually and used directly by man. Thus, there are tremendous possibilities for increasing the volume of available water for use by the public and industry.

If they work together, therefore, the scientist, the administrator, and the citizen-customer can pool their resources for the good of all. The result will be a future that *can* be bright and prosperous if each person will catch a true vision of the importance he plays in mankind's pursuit of freedom and happiness.

A Toast to Water

I hold in my hand a glass of water. I almost said a "simple glass of water," because more just like it can be secured from any one of a thousand or so faucets in this hotel. But it is *not* a "simple glass of water." It is a highly refined manufactured product which bears no more resemblance to the raw material from which it was produced than a bar of finished steel bears to the ore from which it was made. Between the water in the river and the water in this glass our industry has interposed an elaborate manufacturing process and the most efficient system of product distribution yet devised.

But all these material facilities could never have existed without the vision of the men who realized that, unless they were provided with a supply of safe, potable water, our cities could not continue to exist. Nor could these men have produced without the "know-how" of the manufacturers who have given us the equipment and supplies with which to work. But, most of all, water works plants could never be successfully operated without that group of dedicated men who, often at considerable financial sacrifice, stand guard to insure that this glass of water will always be safe and palatable.

So it is that I propose a toast to the members of that great team—those who saw the vision, those who provide the tools, those who use the tools so effectively—and to the citizen-customer, the most important person in the world.

Economics of Stream Flow Regulation

Richard Hazen

A paper presented on Nov. 4, 1955, at the Virginia Section Meeting, Roanoke, Va., by Richard Hazen, Partner, Hazen & Sawyer, New York, N.Y.

THE growing demands for municipal, industrial, and agricultural water supplies in southeastern United States have been emphasized by the recent activity in the legislatures of Virginia, North Carolina, and South Carolina. This activity has included proposals for the establishment of state water policies and far-reaching changes in the present water laws, but, fortunately, the fact that new laws alone will not provide more water is becoming recognized. Although there are areas which have such extensive ground water potentials that they are utilized to only a small degree, the problem in most areas is to find ways of storing flood waters for use in the dry season.

The subject of stream flow regulation, as treated in this article, will be considered mostly as it pertains to the relatively small storage reservoirs required for regulation of low stream flows and for development of water supplies. Flood control projects and hydroelectric projects on main river stems will not be discussed. Such projects are of such dimension as to be considered in an entirely different class. A 15,000-mil gal reservoir on the upper Jackson River, for example, would increase the low flows at Covington, Va., from 45 to 100 mgd. The proposed Gathright Project for flood control and power generation in the same area would require a 257,000-mil gal reservoir and a small reservoir downstream to reregulate the power house discharges. In many areas where large multipurpose reservoirs cannot be justified, small reservoirs for stream control would be extremely useful. How profitable the building of such reservoirs will be depends upon the value of water in the area.

Background

Before considering cost, it seems worth while to review briefly a few basic hydrological factors.

1. In the Appalachian Mountain states, extremely low flows occur for relatively short periods—a few weeks in ordinary years, a few months in extreme droughts. There is no problem of having to store water to carry through dry cycles of several years' duration. The extent of low flows is reflected in the duration curves of the daily flows of the Jackson River at Covington, Va., shown in Fig. 1, and the frequency plotting of the low flows at the Frankstown Branch of the Juniata River in Pennsylvania, shown in Fig. 2.

2. In areas with similar rainfall, runoff, topography, and geology, the stream runoff and storage requirements can be compared on the basis of runoff and storage per square mile of drainage area. Before proceeding on this basis, however, it must be established that the runoff patterns in the areas are truly similar. Two river basins immediately adjacent may

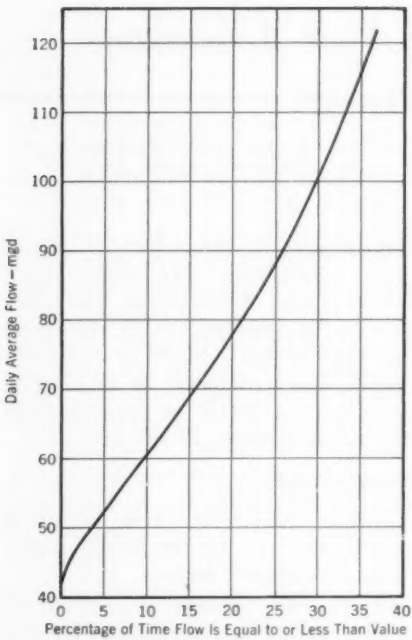

Fig. 1. Duration Curve for Daily Flows of Jackson River, Covington, Va.

The drainage area is 440 sq miles.

have entirely different hydrological characteristics.

3. Some streams have much more natural storage than others. The storage may be in ponds and lakes or in sand and gravel deposits below the ground surface. Natural storage is indicated by occurrence of persistent stream flows even many weeks after a rain. Lack of natural storage results in "flashy streams," with heavy storm runoff and little discharge during the dry season.

4. The artificial storage required to regulate a flashy stream is relatively greater than the storage required for a stream having substantial natural storage. The variation in storage requirements is illustrated by Fig. 3, which shows the mean, maximum, and minimum storage requirements for 28 rivers in the Appalachian Mountain area and fifteen streams in the Midwest. Storage data for the Tar and Neuse Rivers in North Carolina are shown in Fig. 4. The curves in both Fig. 3 and 4 have been adjusted for the release of water downstream for the benefit of lower riparian owners.

5. The amount of storage to be provided in a specific instance depends upon the value of water and the effect of an occasional shortage. A certain reservoir capacity is needed if there is to be complete safety each year. If some deficiency can be tolerated on occasion—for instance, once in 10 years or once in 20 years—the reservoir can be considerably smaller. The variation in storage requirements at the Jackson River for years of different dryness is shown in Fig. 5.

Similar analyses of the Chickahominy River show that, for a yield of 0.4 mgd per square mile, the storage needed in the 98 per cent dry year is 30 per cent greater than the capacity needed in the 90 per cent dry year. For a draft of 0.25 mgd per square mile from the North Branch of the Shenandoah River, the 98 per cent storage requirement is 42 per cent greater than the 90 per cent storage requirement.

This quick review of the hydrological aspects is sufficient to demonstrate that any rational approach to stream flow regulation must begin with an analysis of the stream itself and careful consideration of the purpose for which storage is to be provided.

Costs

The cost of building reservoirs has risen sharply in recent years. The increase has been caused not only by higher construction costs but also by the fact that the best sites were used first,

and the sites remaining are more difficult and costly to develop. Except in a few remote locations, the acquisition of land, the relocation of highways and railroads, and other expenses incidental to the flooding of occupied land have become expensive. Historical costs of projects completed many years ago are of limited use in estimating the cost of new reservoirs. Furthermore, it is almost impossible for any person or organization to acquire sites at reasonable cost without the power of eminent domain.

Because there is little similarity in construction conditions and requirements at reservoir sites, it is impossible to generalize on reservoir costs. Figure 6 shows the estimated cost per million gallons for several proposed reservoirs in Pennsylvania, Virginia, and North Carolina. The reservoirs have not been built, and in some instances only preliminary estimates were made. The data do suggest in a general way the range in costs to be expected under normal conditions, however.

Cost estimates of a 13-mgd water supply at numerous locations in the Appalachian Mountain region were made in connection with a survey of the synthetic liquid fuel potential of the United States. Construction costs of projects with storage, including cost of reservoirs, pipelines, pumping stations, and filter plants, range from $210,000 to $250,000 more, per 1-mgd capacity, than the cost of projects not requiring storage. For a supply of 13 mgd, the reservoirs needed in the region would range in size from 2,300 to 7,900 mil gal, with a median of approximately 4,000 mil gal.

The relationship between reservoir costs and the cost of water may be indicated by examples. Table 1 is an analysis of two storage projects. One

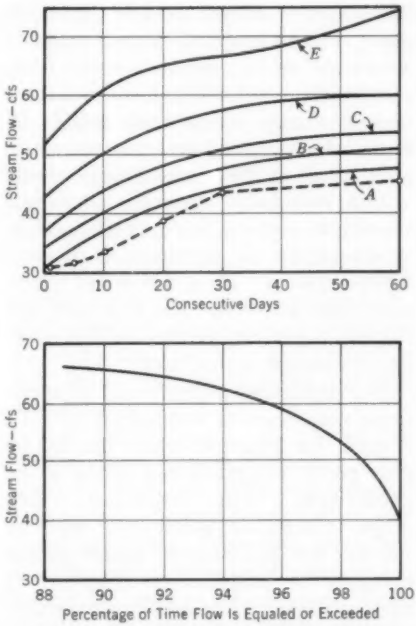

Fig. 2. Flow Curves for Frankstown Branch, Juniata River, Williamsburg, Pa., 1930–48

The top graph shows low-flow frequency curves for the following: A—*98 per cent dry year;* B—*95 per cent dry year;* C—*90 per cent dry year;* D—*75 per cent dry year; and* E—*50 per cent dry year. The dotted line is for 1930, which is the driest period on record. The bottom graph shows a flow duration curve. The drainage area is 295 sq miles, and the mean flow is 388 cfs.*

deals with relatively small storage on a large stream; the other with a fairly high development of a small stream. These data are based upon the flow record of the Jackson River at Covington and the storage curves shown in Fig. 5.

The analysis in Table 1 indicates a cost of water of $13.10 per 1 mil gal or 1.3¢ per 1,000 gal under Project A. The cost under Project B would be

$32.60 per 1 mil gal or 3.3¢ per 1,000 gal. For municipal water supplies where the retail price may range from 20¢ or 30¢ per 1,000 gal, the indicated cost of storage is relatively small. In areas where ground water is not available, or where there are no large rivers within reasonable distance, additional municipal supplies undoubtedly will be developed by the construction of storage reservoirs. Even where the reservoir sites are poor, the cost will not impose a serious burden on the average municipal water user.

For industries using small or moderate quantities of water—up to 1 or 2 mgd, for example—even the $32.60 cost per 1 mil gal is not likely to be prohibitive. For industries using large volumes of water (10 mgd or more), the $32.60 cost would almost certainly be prohibitive. The $13.10 cost is not excessive and is comparable with that paid in many regions.

For other industries, the water might be worth more in the disposal of wastes than as additional water supply. On most streams the degree of waste treatment to be provided is fixed by the extreme low flows during the dry season. For one example studied, it was found that by doubling the low flows the existing waste disposal works could adequately treat 30–40 per cent more waste. While the capital cost of building a reservoir would be substantially greater than the cost of additional treatment facilities, there would be a large saving in operating expense. The benefits to be derived from storage in any particular project can be determined only after thorough study of the waste, the cost of treatment, the quantity of water required for dilution, and the effect of reaeration in the particular stream.

In arid and semi-arid parts of the country, the cost of water to farmers is higher than the cost of storage indicated in the preceding analysis. The farmer's cost usually includes pumping and distribution, and direct comparison is not possible.

Irrigation is comparatively recent in the eastern states and has usually been

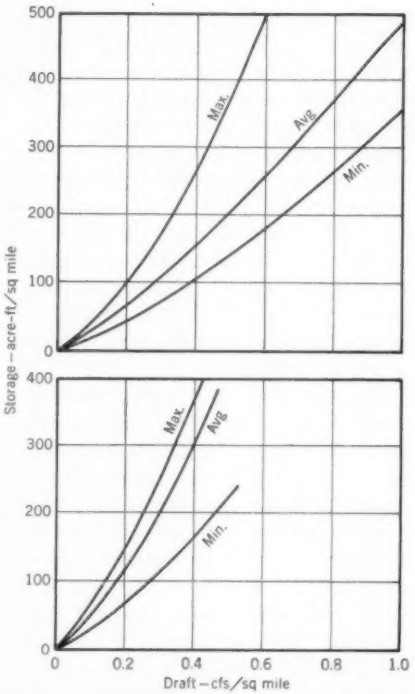

Fig. 3. Storage Curves Based on Driest Period on Record and Release of All Flows Below the 90 per cent Flow

The top graph shows data from 28 Appalachian Mountain Rivers, with a mean flow of 1.52 cfs per square mile; the average, minimum, and maximum at 90 per cent flow were 0.08, 0.02, and 0.18 cfs per square mile respectively. The bottom graph shows data from 15 midwestern rivers, with a mean flow of 0.81 cfs per square mile; average, minimum, and maximum for 90 per cent flow were 0.04, 0.01, and 0.14 cfs per square mile, respectively.

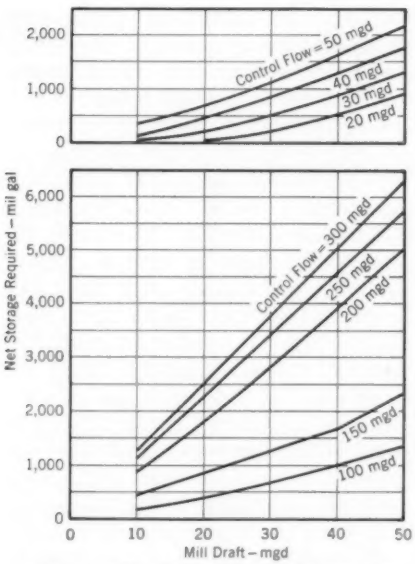

Fig. 4. Net Storage Required at Tar River, Greenville, N.C., and Neuse River, Kinston, N.C.

The top graph is for the Tar River, with a drainage area of 2,620 sq miles, and the bottom graph is for the Neuse River, with a drainage area of 2,690 sq miles. The limiting flow below which water would not be taken from the river for mill purposes is designated as the "control flow."

confined to areas served easily by pumping from wells or nearby streams. The small farm ponds so common throughout the southeastern states will not be adequate in the future, and it seems almost certain that irrigation districts will be formed so that reservoirs of considerable size can be financed.

The reservoirs that have been considered in this paper do not offer much in the way of hydroelectric power generation. One million gallons of water falling 100 ft will generate about 250 kwhr. With power worth 1¢ at the power plant, the daily revenue would be $2.50. If this power were available 24 hr a day, 365 days a year, it might be of considerable value, but reservoirs built for low-flow control must be drawn down during the dry season, and whatever head is available at the dam at such times will be lost.

Examples

Reservoirs for stream flow regulation are not new, and there are several instances of successful operation. In each, the cost of river regulation has been paid for by the principal beneficiaries.

For many years, the Wisconsin Valley Improvement Company, chartered in 1907, has built and operated reservoirs to reduce floods and to augment low flows. The costs have been allocated among 26 public utility and industrial users who benefit from greater and more regular power generation. No charges have been levied for stream pollution abatement al-

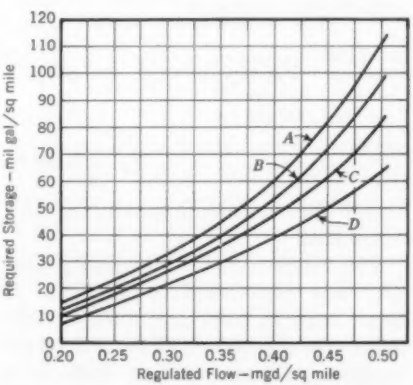

Fig. 5. Storage Yield Curves for Jackson River, Covington, Va.

The drainage area is 440 sq miles. Curves indicate data for the following: A—*98 per cent dry year;* B—*95 per cent dry year;* C—*90 per cent dry year; and* D—*80 per cent dry year.*

TABLE 1

Illustrative Storage Projects

Item	Storage Project	
	A	B
Drainage area—*sq mi*	400	40
Minimum daily flow—*mgd*	40	4
Required firm supply—*mgd*	80	16
Increase in firm supply—*mgd*	40	12
Flow required per drainage area—*mil gal/sq mi*	0.2	0.4
Storage required—*mil gal/sq mi*		
80 per cent dry year	8	39
95 per cent dry year	12	53
98 per cent dry year	14	60
	For 95 per cent Dry Year	For 80 per cent Dry Year
Storage required—*mil gal*	4,800	1,560
Design storage—*mil gal*	5,000	1,700
Estimated cost of reservoir		
Cost per mil gal—$	750	1,600
Total cost—$	3,750,000	2,720,000
Cost per quantity obtained—$/*mgd*	94,000	226,000
Estimated cost of water		
Interest and amortization (4.68 per cent per annum)*—$	176,000	128,000
Operating expenses—$	15,000	15,000
Total	191,000	143,000
Increased firm supply of water—*mil gal/yr*	14,600	4,380
Cost per 1 mil gal—$	13.10	32.60
Cost per 1,000 gal—¢	1.3	3.3

* Equal annual payments, for 40 yr, at 3¼ per cent interest.

though this valuable byproduct is now recognized.

The Hudson River Regulating District was established in the state of New York in 1922 for flood control and river regulation. The 283-bil gal Sacandaga Reservoir was built at a cost of approximately $11,000,000. Interest and amortization on the bonds issued by the district have been paid by the several municipalities receiving flood control benefits and the industries and power companies that use the river for power and water supply. The bonds, which carried interest rates of 4.5–4.7 per cent, are not actual revenue bonds because the assessment against the municipalities and industries are liens on the property. Since the district was formed, however, the indebtedness has been reduced to $3,750,000 and nothing has been paid by the state of New York or the general taxpayer. As noted in the case of the Wisconsin River, stream pollution abatement and the improved quality of

raw water throughout the Hudson River are important benefits, but no direct charges are made for these.

The Savage River Dam on the headwaters of the north branch of the Potomac River was built largely with federal funds but is now operated by the Upper Potomac River Commission. This reservoir, with a capacity of 6,600 mil gal, is used primarily to increase dry-weather flows from about 30 mgd to 60 mgd, thus reducing the effects of industrial pollution in the river in the vicinity of Cumberland. The commission was established by the state of Maryland, but all operating expenses are borne by the upriver counties benefited. The large industries in the region are substantial taxpayers in these counties and contribute a large part of the cost.

The Miami Conservancy District in Ohio, known widely for its pioneering work in flood control, has paid off its entire indebtedness and is now studying the feasibility of building additional reservoirs for regional water supply purposes and low-flow control.

In spite of the fairly high construction costs, reservoirs will provide additional water supplies in southeastern United States at a much lower cost than such other methods as distillation or demineralization of brackish waters. These other methods, described occasionally in the press, may be applicable in some arid areas, but their cost would be many times greater than the cost of reservoirs.

Conclusion

Reservoirs cannot be built economically by private corporations or small communities, and, as the demand for storage increases, construction by public or semipublic water districts seems certain. Such districts should have the power of eminent domain; under present tax laws they can borrow money at low rates. Legislation would be required in many states to permit the establishment of such districts, but public opinion is generally favorable. In some areas, the demand for water is such that a district formed to develop the water resources could arrange in advance for guaranteed annual payments by the municipalities and industries to be benefited. These guarantees would serve as the basis for financing revenue bonds. In areas where the immediate revenue would not finance the entire cost, it may be advantageous to the community to underwrite some of the expense in order to encourage its industrial development. Except where there are natural ground water resources, the prospects of developing additional water supplies by methods less costly than storage are poor in most eastern states. Communities should band together and join with industry to develop these resources now, while the necessary sites are still available.

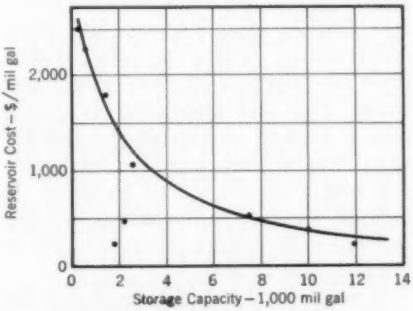

Fig. 6. Approximate Reservoir Costs, Appalachian Mountain Area

Data are based on estimates made for proposed reservoirs.

Futility of Barriers in the San Francisco Bay System

Raymond A. Hill

A paper delivered on Oct. 28, 1955, at the California Section Meeting, Sacramento, Calif., by Raymond A. Hill, Partner, Leeds, Hill & Jewett, Cons. Engrs., Los Angeles, Calif.

IN 1953 the author was one of many who accepted the premise that some barrier should be built across San Francisco Bay for salinity control and water conservation. There had been, at that time, so much discussion about the Reber Plan and alternative proposals that it seemed as though the controversy was merely a matter of where and how a barrier should be built. It had not occurred to the author that the premise itself—that a barrier *should* be built—might be false.

It was with this background that the author accepted the chairmanship of the board of consultants appointed by the Water Project Authority of the State of California, pursuant to the Abshire-Kelly Act of 1953 (*1*). It is the author's belief that similar views were held by the other members of the board: Hans Kramer, Malcolm Pirnie Sr., Charles E. De Leuw, and Philip C. Rutledge. Nevertheless, after more than a year of intensive study, the board came to the unanimous conclusion that construction of any barrier across San Francisco Bay, or any of its arms (*see* Fig. 1), would be futile because the primary purposes would not be realized, while the correlative benefits could be gained by other means.

The background for this conclusion is set forth in the official report of the board transmitted to the California State Legislature in March 1955. The purpose of this paper is to explain why construction of any closed barrier across San Francisco Bay would be a waste of money.

In the first place, the real purposes of a barrier between the fresh waters of the Sacramento-San Joaquin river system and the salt waters of the Pacific Ocean have been obscured in the controversies over land reclamation, transportation, and other matters foreign to the original issue. Construction of a barrier is not a prerequisite to land reclamation; on the contrary, land adjacent to San Francisco Bay has been reclaimed whenever there was economic justification for so doing. Much of the business district of San Francisco, for example, is on reclaimed land. Rail and highway transportation around and across San Francisco Bay has been provided as the need developed; better transportation does not depend upon the construction of a barrier or any system of barriers.

In brief, if there is economic justification for reclamation of tidelands in the San Francisco Bay region, those interested should finance such work upon a sound basis. If there is need for more transportation facilities, they should be provided in the manner in which such facilities have heretofore been provided. Such correlative mat-

ters should be separated from the primary purposes of a barrier—salinity control and water conservation.

Salinity Control

The need for salinity control arose from the fact that there are about 400,000 acres of highly developed agricultural land in the Sacramento–San Joaquin Delta. The water supply for irrigation of these lands is obtained from both the Sacramento and San Joaquin rivers and from the many channels that divide the delta into islands. At times of flood, the river channels and the interior channels are swept free of all salt water, but during the summer and fall of each year, when there is little flow in any of the rivers entering the delta, tidal action carries salt water upstream from Suisun Bay, the most easterly arm of San Francisco Bay.

Diversions of water for irrigation in the Sacramento and San Joaquin valleys progressively depleted the natural flow of the rivers entering the delta until there was not enough fresh water left to repel the salt water. The city of Antioch, located near the lower end of the delta, then brought legal action to force the delivery of more water down the Sacramento River during the months of low flow. Consequently, when the Central Valley Project of California was being planned, prevention of the intrusion of sea water into the delta region was a prime consideration.

The solution then adopted was the impounding of flood waters in upstream reservoirs and the release of enough water from these reservoirs in the dry months of each year to repel the saline waters of Suisun Bay from the channels of the Sacramento and San Joaquin rivers, and thus from the interior channels of the delta. This procedure has been followed and must continue to be followed until some other means of salinity control is provided.

Repulsion of sea water is not enough, however, to provide full salinity control. Surplus waters of the Sacramento River are carried across the delta in natural channels, and are then pumped into a canal for delivery into the San Joaquin River near Mendota in exchange for water diverted out of that river further upstream for use in the southerly portion of the Central Valley. This water, in its transit across the delta from the Sacramento River to the pumping plant near Tracy, is polluted by irrigation return flows and rising connate waters. The amount of salt thus added to the supply from the Sacramento River is so large that there are times each year when water delivered into the San Joaquin River does not meet the standards of quality specified in the agreement under which this exchange of water is made. This has been embarrassing—to say the least—to the Bureau of Reclamation of the US Department of the Interior.

Hence, if only repulsion of ocean water from the delta were involved, almost any barrier plan would satisfy the requirements of salinity control. However, a barrier cannot prevent deterioration of the waters of Sacramento River carried across the delta for use there and in other areas farther south; a barrier would only aggravate the present situation by trapping all of the salts brought in by irrigation return flows and rising connate waters, some of which are very saline.

Water Conservation

No one should question the eventual need for conservation of the large vol-

umes of fresh water that now flow into San Francisco Bay during the winter and spring of each year; the timing and method of such conservation efforts are, however, other matters.

Under natural conditions the average outflow into San Francisco Bay was in excess of 30,000,000 acre-ft per year. This average, however, is meaningless: even before there were any upstream reservoirs, and before diversions were made from the river for irrigation and other purposes, there were wet years in which great floods occurred and dry years in which much less than the average outflow wasted to the ocean. Furthermore, during the summer and fall months of each year, the natural flow of the Sacramento and San Joaquin rivers was small compared to the runoff during the winter and spring.

There are now many storage dams on the rivers of the Sacramento–San Joaquin system; these hold back much of the flood waters that formerly reached San Francisco Bay. In addition, the diversions of water for irrigation of land and other purposes now exceed the natural runoff in the dry months of each year, requiring the release of water from these upstream reservoirs.

There is no justification for the assertion frequently made that more than 30,000,000 acre-ft of water per year now wastes to the ocean. There is even less basis for the claim that 10,000,000 acre-ft of water could be conserved each year by the construction of barriers in San Francisco Bay. A few of the facts disproving such claims will be considered here.

The total surface inflow to the Sacramento–San Joaquin Delta averaged only 12,300,000 acre-ft per year during the 6-year period October 1928–September 1934. Consumptive uses of

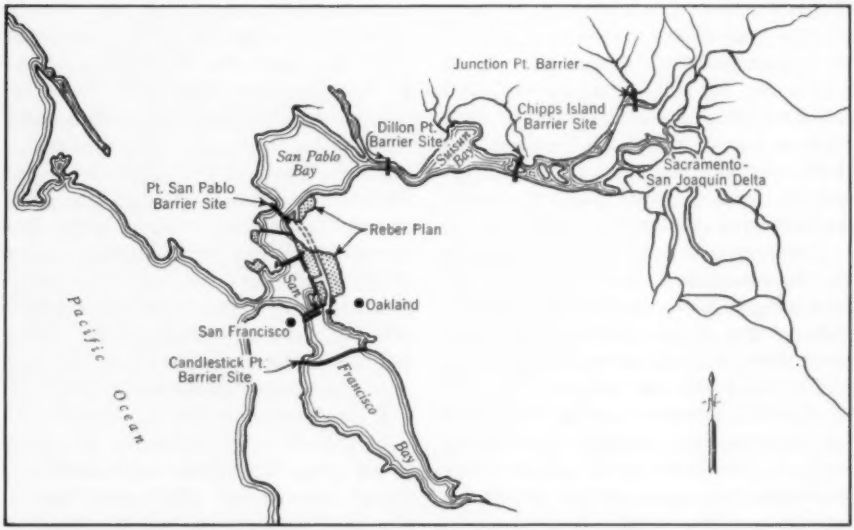

Fig. 1. Proposed Barrier Sites, San Francisco Bay Area

this water were in the order of 1,300,000 acre-ft per year. The waste of fresh water into Suisun Bay thus averaged only 11,000,000 acre-ft per year. This was the quantity available historically during an extended drought for conservation in a barrier pool or by other means.

Since then, Shasta Dam and other storage works have been built. The effect of these has been computed by the state Division of Water Resources (*2*). The quantities of water that would still have wasted in these years into Suisun Bay under present conditions of development upstream is shown in Table 1. Even if all other inflow to San Francisco Bay were added, the total waste of fresh water would average less than 8,000,000 acre-ft per year during a similar period of 6 consecutive dry years. This is only about one-quarter of the waste that occurred before use was made of the waters of the rivers entering the bay.

TABLE 1

Inflow to Suisun Bay After Regulation in Existing Reservoirs, 1928–34

Years	Inflow *acre-ft*
1928–29	5,654,000
1929–30	10,589,000
1930–31	3,987,000
1931–32	11,665,000
1932–33	5,913,000
1933–34	6,299,000
Avg	7,334,500

The time of occurrence of floods in the Sacramento and San Joaquin rivers is also of particular significance in the consideration of a barrier as a means of water conservation. Such floods occur only in the winter and spring of each year. During the summer and fall of each year the fresh water now entering Suisun Bay is limited to irrigation return flows and releases of stored water for salinity control. The quantities of water given in Table 1 include water released from upstream storage reservoirs to maintain a residual outflow from the delta of 3,800 cfs for salinity control.

If a closed barrier were built at any point between the mouth of Sacramento River and the Golden Gate to prevent the salt waters from the ocean mixing with the fresh waters in the river channels bordering the delta, there would obviously be no purpose in continuing such releases of water from upstream reservoirs for salinity control. The quantity of fresh water that would reach Suisun Bay under such conditions would be only that portion of the river flows which could not be diverted for use in the valleys of the Sacramento and San Joaquin rivers, or within the delta itself.

The supply available for conservation in a barrier pool, therefore, would consist only of those flood waters which could not be controlled by storage in existing reservoirs in the Sacramento–San Joaquin River system, or which could not more profitably be conserved in new reservoirs, such as Oroville Reservoir on the Feather River. Yet John M. Reber, in a signed article on the editorial page of the *San Francisco Chronicle* (*3*), has stated:

> The Reber Plan would create two huge fresh-water lakes having a capacity of an estimated 10,000,000 acre-ft of water. But the "secret" of the water conservation and distribution is that water would be running into these lakes every second of every year.
>
> The plan, therefore, would be to "take out" during the "inflow." In other words, these fresh-water lakes would act

exactly as does a "watering trough." You need one-third of the annual inflow to supply all of California's needs. The remainder just keeps flowing out through the Golden Gate. Water would be flowing over the barrier all the time, year in and year out. The consultants did not understand this at all. They said the water which could be saved would be what could be stored in the lakes between midtide and high tide.

In view of this statement, it would be well to consider a few more facts. The surface area of all of the pools that could be formed under the Reber plan would be about 250,000 acres. In order for the effective capacity of the lakes involved to be 10,000,000 acre-ft, there would have to be a 40-ft range in water level. This would require that the barrier pools be emptied at the end of a dry season and that the city of Sacramento be submerged during floods.

The highest level to which water could be stored without inviting inunda- of the Sacramento–San Joaquin Delta during floods would be maximum high tide level, about 4 ft above mean sea level. The lowest level to which the water could be drawn down without serious interference to navigation would be mean sea level. The conceivable range would thus be only 4 ft, not 40 ft. This would provide 1,000,000 acre-ft of effective storage in these lakes, not 10,000,000—a mere matter of a floating decimal point.

In the opinion of the board of consultants, the practical limit of fluctuation of water levels in any barrier pool would be about 3 ft, and the board felt that experience in operation would demonstrate the desirability of less fluctuation in order to avoid adverse effects on navigation and to minimize the flood control problem in the delta.

It should be obvious, therefore, that there is little opportunity for conservation of water behind any system of barriers because of the limited storage capacity that could be used for storage of flood inflows and because the inflow would be small during the summer and fall of each year. Furthermore, evaporation losses from the surface of a barrier pool during the summer and fall would alone cause a lowering of more than 2 ft and there would also be losses of fresh water through locks and fish ladders. Hence, the conclusion reached 20 years ago, that storage could be provided in upstream reservoirs more economically than in a barrier pool, is still valid.

Quality of Water

Even if the question of the quantity of water that could be conserved by construction of a barrier or system of barriers in San Francisco Bay is ignored, there remains the question of the quality of that water. Unless the water so conserved will be usable for all purposes at all times, nothing would be gained.

It is not sufficient for the proponents of barriers to state that the water is pure when it flows off the Sierra Nevada, and that all that is necessary is to control pollution of the water between the mountains and the barrier pool. The author doubts that the State Water Pollution Control Boards of California would care to undertake the task of preventing deterioration of the quality of the water, because they are accustomed to dealing with physical facts.

It is inevitable that the pure waters leaving the mountains will not remain pure for the following reasons:

1. Much of the water will be used for the irrigation of land, and the return flows will contain substantially more salts than the natural water.
2. Some of the water will be used for domestic and industrial purposes, resulting in greater concentrations of soluble salts.
3. Some of the inflow to Suisun Bay is now saline connate water that cannot physically be intercepted.
4. There will be some transfer of sea water through navigation locks.
5. Salt now in the deposits underlying the present salt water bays will diffuse upward into the overlying fresh water in any barrier pool.

The combined effect of all these factors would result in the impounded water becoming too saline for many uses during an extended drought when there would be little, if any, outflow through the spillways at the barrier.

The quality of the water must also be considered, of course, from the standpoint of organic pollution. Such pollution could come from the disposal of sewage from the many communities in the Bay Region even though every reasonable precaution were taken; it could also come from the wastes of industrial plants. Furthermore, conditions would be favorable to the growth of algae because any barrier pool would be relatively shallow and the temperature of the water would approach the mean daily air temperature toward the end of each summer.

It is unnecessary to emphasize the risk of organic pollution: it requires little imagination to visualize the water in a barrier pool becoming foul.

Alternatives

Fortunately, the futility of bay barriers as a means of conserving water of suitable quality is not of serious moment. There are alternative and, indeed, better solutions to the problem.

It is doubtful that anyone now seriously questions the wisdom of the construction of Shasta Dam in lieu of a barrier when this decision was made 20 years ago; nor is it likely that anyone seriously questions the desirability of the construction of other dams that have since been built on the Yuba, American, San Joaquin, and Kings rivers. It is the author's conviction that 25 years from now the wisdom of constructing Oroville Dam on the Feather River will not be questioned. Much more effective storage can be provided in this and other proposed reservoirs than could possibly be made available for water conservation under any barrier plan.

There is also the alternative plan suggested by Cornelus Biemond, the renowned Dutch engineer, who studied the problem last year. Biemond proposed, in effect, that the fresh and relatively pure water of the Sacramento River be conveyed across the delta in a separate channel, thereby preventing any pollution or contamination of the fresh water by sea water or by irrigation return flows or rising connate waters. The water supply for the delta lands would be taken from this separate channel so that there would no longer be need for the release of water from upstream reservoirs for salinity control. The polluted return waters and the saline connate waters would not be intercepted; they would flow into Suisun Bay and be carried out to the ocean by tidal action, just as industrial wastes and sewage are now disposed of naturally.

In brief, under this alternative plan, the fresh water would not be permitted

to commingle with any less desirable water; under any barrier plan, the so-called "fresh-water" lake would trap all waters of undesirable quality regardless of their origin.

The Biemond Plan was recommended by the board of consultants because it would best accomplish the real purposes of salinity control and water conservation. The board could not recommend the construction of a bay barrier because any barrier lake would be the cesspool of the great Central Valley of California.

References

1. *The Abshire-Kelly Salinity Control Barrier Act of 1953.* Chapter 1104, Statutes of 1953, Calif. State Legislature, Sacramento, Calif.
2. Report to the Water Project Authority of the State of California on Feasibility of Construction by the State of Barriers in the San Francisco Bay System. Div. of Water Resources, State of Calif. Dept. of Public Works, Sacramento, Calif. (Mar. 1955). p. 54.
3. REBER, JOHN M. Signed editorial. *San Francisco Chronicle.* p. 2 (May 29, 1955).

Correction

In the paper, "Some Novel Approaches to Population Studies," by K. L. Kollar (April 1956 JOURNAL), the integration on p. 430 is in error. The terms $\left(\frac{1}{bx}\right)$ and $\left(\frac{1}{br}\right)$ should read, respectively: $(1 + bx)$ and $(1 + br)$.

Chlorination of Deep Reservoirs for Taste and Odor Control

Ray L. Derby

A paper presented on Oct. 27, 1955, at the California Section Meeting, Sacramento, Calif., by Ray L. Derby, Prin. San. Engr., Dept. of Water & Power, Los Angeles, Calif.

THE city of Los Angeles operates sixteen major reservoirs in Southern California. These vary in capacity from approximately 100 acre-ft to more than 36,000 acre-ft and in maximum depth from 25 to 130 ft.

The summer stratification of these deeper reservoirs often curtails the use of the deep waters and creates taste and odor problems in the late summer and fall. These problems become especially troublesome in the older reservoirs where existing outlet structures are not versatile enough to permit avoidance of the water strata which have become objectionable. Two such reservoirs which have required special attention are those at Encino and Lower Hollywood. Pertinent data for these reservoirs are shown in Table 1.

Figure 1 shows typical conditions at the Lower Hollywood reservoir. Under some high-flow operation it becomes necessary to use three or four of the five gate elevations. The bottom gate at el 580 (not shown) is buried under many years' accumulation of silt and organic debris and cannot be used. The graphs in the center of the chart show temperature and depth relationships at different periods of the year. The circulating zone, or epilimnion, gradually increases in temperature throughout the spring and summer, resulting in the formation of a relatively narrow stratum characterized by sharp temperature differentials. This layer is called the transition zone, mesolimnion, or thermocline, and it effectively prevents the stagnation zone, or hypolimnion, from replenishing its oxygen supply.

The hypolimnion is the problem zone. Shut off from a source of DO by the thermocline, the hypolimnion suffers oxygen depletion, which results in the anaerobic decomposition of organic debris such as dead algae, aquatic weeds, leaves, and dead fish. This causes tastes and odors, produces a more acid environment, promotes sulfide formation, and results in increased ammonia, phosphates, nitrogen, and various metallic constituents.

Table 2 shows progressive changes in water quality during stratification at the Lower Hollywood reservoir. The tremendous increase in the products of anaerobic decomposition below the thermocline is readily noted. These changes deteriorate water quality because excessive tastes and odors will enter the circulating zone during the fall equalization. In addition, the high ammonia and hydrogen sulfide content will require several times the normal chlorine dose in order to maintain a free chlorine residual in the distribution system. Increases in phosphates, nitrogen, and ammonia also add to the already burdensome algae problems.

Experimental Aeration

In recent years the buildup of organic material in the Lower Hollywood and Encino reservoirs has been so fast that it has become necessary to reduce tastes and odors originating under anaerobic conditions near the reservoir bottoms. This problem was first attacked in 1953, when compressed air was used at both reservoirs. At that time an attempt was made to lift the oxygen-deficient bottom water by air entrainment. Water was moved in large volumes but it tended to fall back to the deep zone. Insufficient horizontal circulation prevented complete success, but the thermocline was definitely lowered in both reservoirs.

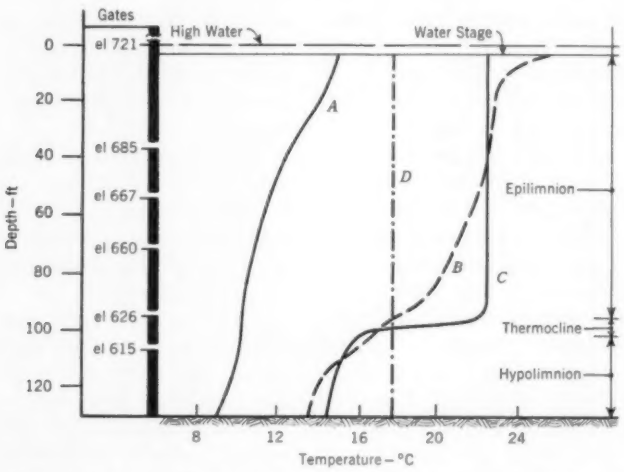

Fig. 1. Lower Hollywood Reservoir Temperatures, 1953

Curves indicate temperature changes at various stratifications for following days of test: Curve A, *Mar. 5; Curve* B, *Jul. 16; Curve* C, *Aug. 26; Curve* D, *Nov. 6.*

Chlorination at Lower Hollywood

Chlorination of the hypolimnion at the Lower Hollywood reservoir was tried in 1954. Gas-feed equipment was first used because gas diffuses through a much greater depth of water than does solution feed. It was soon found that high back pressure at the depth treated (130 ft) caused occasional liquefaction of the gas, thus rendering the use of plastic or rubber tubing impossible. Even copper tubing would not hold up at the water line, where the liquefaction apparently occurred. Some success was finally achieved by feeding both gas and liquid chlorine through $\frac{1}{4}$-in. black-iron pipe to a grid located about 2 ft above the bottom of the reservoir. The bulk of the gas tended to go into solution 10–25 ft above the bottom, however, and it was decided to experiment with solution feed in 1955.

Figure 2 shows details of the 1955 chlorine solution feed line at Lower

Hollywood. The suction line extended 130 ft to the bottom of the reservoir, and the discharge line to the same depth about 350 ft distant from the suction line. It was hoped that by drawing off water of the same density and temperature as that at the point of discharge the chlorine solution would diffuse in the bottom area where it was most needed.

TABLE 1

Capacity Data for Encino and Lower Hollywood Reservoirs

Item	Encino Reservoir	Lower Hollywood Reservoir
Capacity—*acre-ft*	3,000	4,000
Area—*acres*	80	82
High water to bottom gate—*ft*	63	106
High water to present bottom—*ft*	97	130
Volume in hypolimnion* —*acre-ft*	100	120

* Maximum at start of season.

Both of these lines are 1½-in. high-impact strength polyvinyl chloride plastic. The discharge line is capped and the last 15 ft of plastic pipe is drilled with 30 ¼-in. holes to facilitate diffusion. After considerable experimentation it was found necessary to make the suction and discharge lines adjustable so that water of optimum temperature could be taken from or discharged into any stratum. Recent experiments indicate that it may not always be necessary to use the cold bottom water for the suction line. Using high-pressure water with a temperature 10°C higher than the stratum into which it was discharged, it was possible to keep the chlorine near the bottom. This indicates that the diffusion at the discharge outlet plus the increase in density caused by the solution of chlorine was sufficient to counteract the temperature differential.

Absolute control of chlorine concentrations in the hypolimnion is not possible under the conditions described. An average feed of 200 lb per day was started. Concentrations of 0–300 ppm in the bottom zones were observed. Concentrations in strata within 12 in.

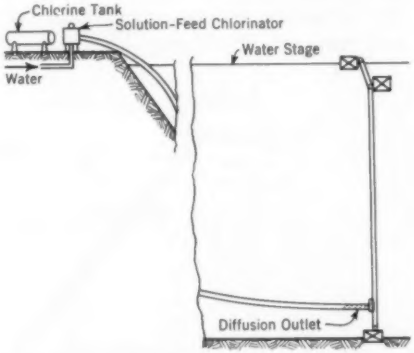

Fig. 2. Deep-Water Chlorine Solution Feeder, Lower Hollywood Reservoir

Discharge line from chlorinator to diffusion outlet (at a depth of 130 ft) consists of 350 ft of 1½-in. polyvinyl plastic tubing. The last 15 ft of this tubing is drilled with 30 ¼-in. holes to facilitate diffusion. Height of suction for discharge lines may be adjusted by manipulation of double plastic clothesline wire looped through anchor ring and float.

of each other varied as much as 50 ppm and remained that way for several days. At one time difficulty was experienced getting chlorine into the thermocline, where small amounts of sulfide persisted. Regardless of the elevation at which the chlorine was introduced, it would end up either below or above the sulfide. Even the introduction of a ton of liquid chlorine did not eliminate this "sulfide sandwich." It was finally

overcome by increasing the solution feed to 400 lb per day and building the concentration up so high (150 ppm) in the hypolimnion that diffusion finally "overpowered" the trace of sulfide. In spite of the difficulties encountered in diffusing the chlorine, the overall results were quite encouraging.

Table 3 shows the chemical changes in the hypolimnion before chlorination in 1953 and after chlorination in 1955. The reduction in decomposition products was very gratifying.

lems caused by liquefaction of the chlorine gas have been encountered at this depth.

Figure 3 shows the details of this installation. It consists of a gas-feed chlorinator feeding through 400 ft of ¼-in. standard-strength black-iron pipe terminating in a 10-ft polyvinyl plastic diffuser line. It was expected that the sloping line extending approximately 400 ft would operate satisfactorily; it clogged, however, during the first week. Since it was dismantled and

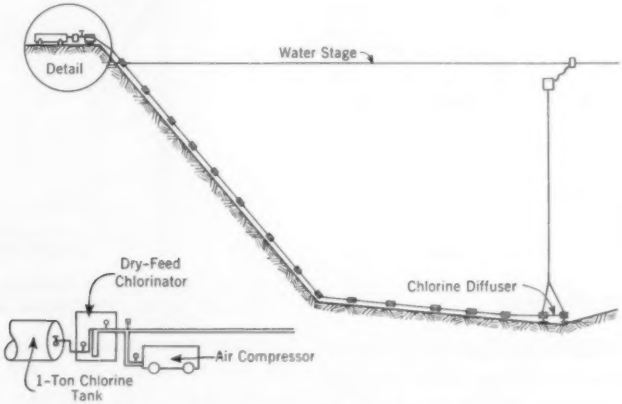

Fig. 3. Chlorine Gas Feed at Encino Reservoir

The chlorine diffuser, protected from bottom mud by load supports, is attached by plastic-covered wire clothesline to submerged rubber float and marker. Chlorine gas is piped from feeder unit (shown in detail) through 400 ft of ¼-in. black-iron pipe which is kept from bottom by 3-ft wooden strips wired to joints. Diffuser consists of 10 ft of drilled polyvinyl chloride plastic tubing.

Chlorination at Encino

Because of the relative simplicity of gas-feed chlorination and also because high-pressure water and power were not readily available, it was decided to try gas feed first at Encino. The dose averaged 100 lb per day and was applied at a depth of 85 ft. No prob-

cleaned it has been possible to keep the line in operation for 5 months by blowing it out with compressed air whenever excessive back pressure is noted. Water is first allowed to enter the pipeline. This dissolves the ferric chloride that accumulates in the line. Air pressure then clears the line. This is re-

peated several times and the line returned to service. Following this procedure it has been possible to maintain a good chlorine residual in the hypolimnion since June 1955 with no sulfide formation. Chlorination was stopped in October 1955. From this single season's experience it appears that the line will last at least 4–5 months with this type of operation.

Figure 4 shows typical conditions at Encino during chlorination. It will be noted that anaerobic conditions are prevented for the entire depth by either DO or chlorine. This chart also demonstrates the problems which can be caused by inadequate gate elevations. Encino has only three gate elevations, the lowest of which (921 ft) is buried in mud. During the late summer the top gate is out of water, so the 955-ft gate is the only one that can be used. As can be seen on the chart, if chlorination is not used there is danger of complete oxygen depletion in the area of the gate. This will result in sulfide formation, or at best delivery of a water so low in DO that problems will develop in the distribution system. When chlorination is used, it requires constant surveillance to make certain excessive chlorine is not drawn into the distribution system.

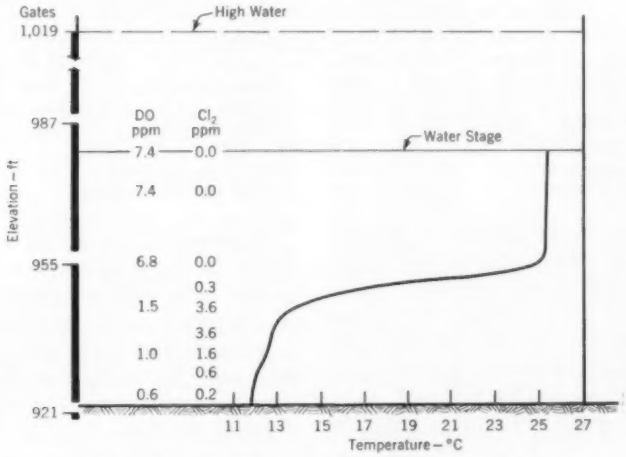

Fig. 4. Effect of Chlorination of Bottom Waters, Encino Reservoir

Data and curve shown are for date of Sep. 15, 1955. Anaerobic conditions are prevented for the entire depth by either DO or chlorine concentrations.

Conclusions

Although chlorination of deep summer-stratified waters is still in the experimental phase, current results have been encouraging. Tastes and odors caused by anaerobic decomposition in the hypolimnion have been greatly reduced and it is believed that the difficulties encountered and the ex-

perience gained will provide the basis for more satisfactory results in 1956. In spite of some measure of success in solving the problems presented, it should be kept in mind that this type of treatment is only a palliative measure and can never completely remedy fundamental deficiencies in the design and operation of the reservoirs themselves. It is apparent that far-sighted planning should provide sufficient gate area at enough elevations to prevent or avoid problems associated with summer stratification. The lowest gate is the most important in this problem. It should be located within a few feet of the bottom and be of sufficient capacity to remove the entire potential hypolimnion before oxygen depletion can take place.

TABLE 2

*Water Quality Deterioration During Stratification—Lower Hollywood Reservoir, 1953**

Item	Apr. 11	Jun. 9	Oct. 2
	Concentration—*ppm*		
DO	9.1	0.0	0.0
Sulfide (S^{--})	0.0	2.8	10.1
Ammonia (NH_3)	0.2	3.8	13.0
TKN†	0.6	4.8	15.1
Phosphate (PO_4)	0.4	2.7	5.8

* Samples taken at el 590.
† Total Kjeldahl nitrogen.

TABLE 3

*Chemical Changes in Hypolimnion—Lower Hollywood Reservoir, 1953–55**

Item	Sep. 1953	Sep. 1955
	Concentration—*ppm*	
Ammonia (NH_3)	13.0	0.05
TKN†	15.0	1.2
Phosphate (PO_4)	5.8	3.3
Sulfide (S^{--})	10.0	0.0
Chlorine (Cl_2)	0.0	80.0

* Samples taken at el 590.
† Total Kjeldahl nitrogen.

In general, gates should be closer together in the deeper zones to provide greater versatility and choice of water. In reservoirs subject to seasonal drawdown, relatively fewer gates are necessary near the surface as they will be too close to the surface or out of water much of the time.

Method for Collecting and Filtering Water Samples Without Airborne Contamination

Norvel M. McClung

A contribution to the Journal by Norvel M. McClung, Prof. of Botany, Univ. of Kansas, Lawrence, Kansas. *The investigation was supported by Research Grant E-153 (C4), National Inst. of Health, USPHS.*

IN studies of the microflora of media in which the number of microorganisms is low, it is necessary to collect and concentrate samples without contamination by airborne organisms. A simple and effective method was devised for the collection and concentration of microorganisms in finished water from city water systems in a study of the survival of fungus spores in this medium. This method, which could be adapted for the collection and concentration of microorganisms in other media, will be briefly described.

Four-liter pyrex serum bottles served as collecting vessels. They were prepared as shown in Fig. 1, and sterilized by autoclaving.

A pipet cleaner, which served as a manifold, was fitted with a rubber hose and a slip-on connector and similarly sterilized by autoclaving (*see* Fig. 2).

The water faucet at the collecting site was sterilized, after flushing, by flaming with an alcohol torch. The rubber tubes from the collection bottles were then connected to glass tubing outlets on the manifold, as shown in Fig. 3. The rubber slip-on connector was fitted to the faucet, and the water was collected. In this way as many as 24 collections could be made simultaneously. Whenever bottles did not fill at the same rate, they were clamped off as they became completely filled.

A modification of the molecular filter membrane apparatus for hydrosols * was made to permit filtration of water without contamination by airborne organisms. As illustrated in the left half of Fig. 4, the cover of the hydrosol unit

* Filters and holders used were manufactured by the Millipore Filter Corp., Watertown, Mass.

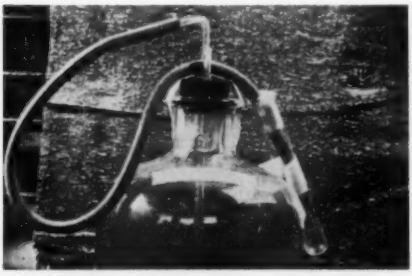

Fig. 1. Sterile 4-Liter Pyrex Serum Bottle

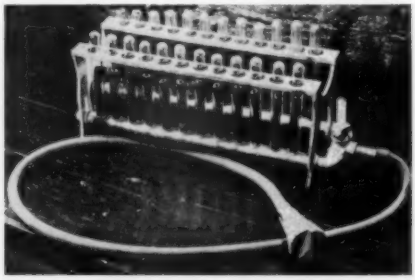

Fig. 2. Pipet Cleaner as Collecting Manifold

(ordinarily used for field sterilization) was adapted for the present purpose by inserting brass screws in the holes, and soldering a 3-in. copper tube in the center opening. In the model shown in the right half of the figure, a lucite cover was made to fit the top of the filter unit, and a glass rod inserted to permit the entrance of water.

After the filters and covers were sterilized by autoclaving, the covers were put in place and sealed with masking tape. Filtering was accomplished by attaching delivery tubes from the sample bottles to the tube passing through the filter cover, and siphoning the sample into the filter by means of a vacuum pump. A clamp was used on the delivery tube to control the rate of the inflowing sample according to the filtering speed. By changing the filter membranes in a hood, aerial contamination could be eliminated at this step.

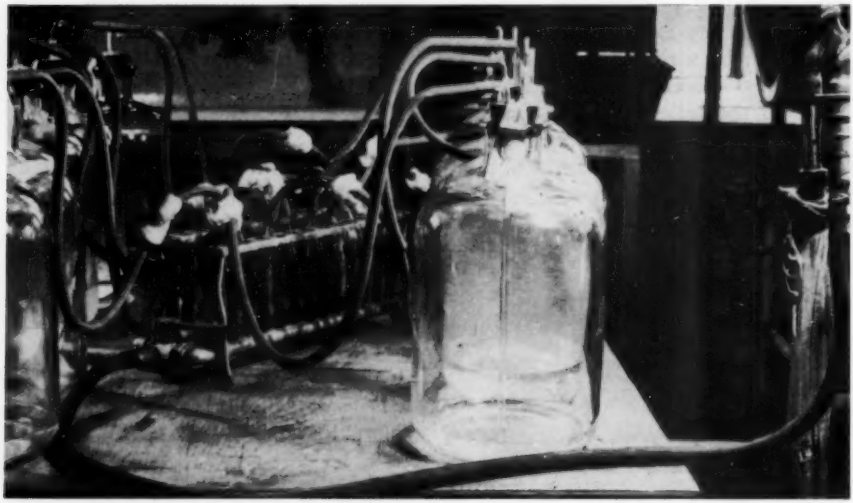

Fig. 3. Apparatus Arranged for Collecting Twelve Samples Simultaneously

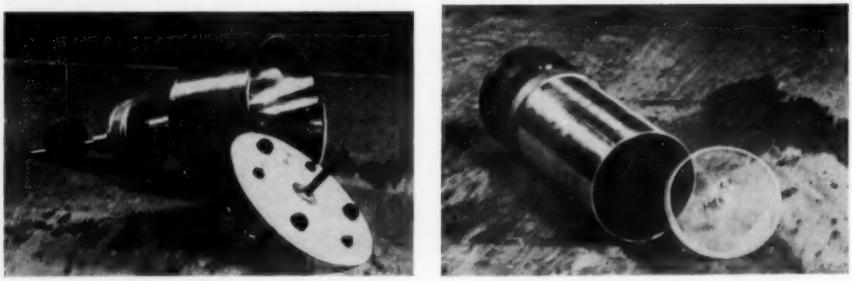

Fig. 4. Hydrosol Filter Units With Airtight Lids

Acid Pretreatment in Aluminum and Fluoride Determinations

William R. Jones and Bert W. Clark

A contribution to the Journal by William R. Jones, Chemist and Bacteriologist, and Bert W. Clark, Chemist and Bacteriologist, both of the Water Dept., Minneapolis, Minn.

DURING the course of a series of fluoride determinations on Minneapolis finished water, discrepancies were noted in the results when the procedure given in *Standard Methods* (*1*) was used for determining the aluminum correction. Investigation disclosed that a better correlation of the data could be obtained when the aluminum sample was initially treated with acid. Good results prevailed when a 50-ml aluminum sample was treated with an excess of 0.2 or 0.3 ml of 0.1 *N* sulfuric acid beyond the total-alkalinity or methyl orange endpoint.

Because of the additional solubility of aluminum caused by the presence of acid, it is important in certain situations to know the amount of acid-reactive aluminum in a sample rather than the concentration of neutral-reactive aluminum as given in the standard procedure. The influence of aluminum in the fluoride determination represents an illustration in point. The same would be true of other determinations performed under acid conditions in which aluminum might have a significant positive or negative effect. Sample acidification has been found useful in the determination of aluminum in Minneapolis water. A rise in the pH of Minneapolis finished water is associated with increased aluminum residual. The stability of the aluminum residual may be limited, as a clear sample of plant effluent having an initial pH of about 8.4 or more often gives a visible alum floc approximately 20 min after treatment with carbon dioxide.

The aluminum procedure in *Standard Methods* (*1*) fails to determine such insoluble aluminum forms as, among others, alum floc or complex aluminum silicates. Obviously, when acid is applied to a sample containing alum floc, the floc dissolves, yielding higher aluminum values than those obtained by the standard colorimetric procedure. It should be emphasized here that aluminum in forms other than alum floc can conceivably dissolve under the specified acid conditions.

Experimental Details

The aluminum values in this study were obtained by visual comparison with standards made up as follows: About 50 ml of distilled water in each short-form (165 mm) 100-ml Nessler tube was mixed with 0.2 ml of 0.1 *N* sulfuric acid. An appropriate amount of standard aluminum solution containing 0.880 g per liter (g/l) of $AlK(SO_4)_2 \cdot 12H_2O$ was then added. The contents of each tube were mixed and allowed to stand at least 1 hr. The standards were then made up to the 100-ml mark with distilled water, treated with 5 ml of acetate buffer solution (see *Standard Methods* for

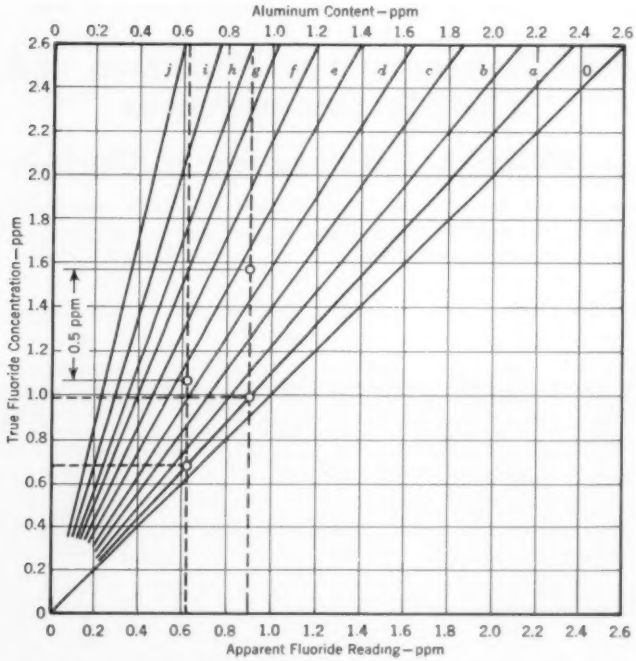
Aluminum Content—ppm
0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6
j i h g f e d c b a 0
True Fluoride Concentration—ppm
0.5 ppm
Apparent Fluoride Reading—ppm

Fig. 1. Determining Fluoride Concentration in Presence of Aluminum in Water

Correction curves such as these can be constructed only if aluminum is the only interfering substance. The aluminum must first be stabilized by acid pretreatment.

the preparation of this solution and the aluminon reagent) from an automatic pipet, and mixed. Aluminon indicator, delivered by a 2-ml automatic pipet, was then mixed with the contents of each tube, and comparisons were made at the end of 20 min. The aluminum standards prepared in this manner were found to be reproducible.

The dilute "working standard solution" recommended in *Standard Methods* is satisfactory only when freshly prepared. This fact was ascertained by preparing three aluminum stock solutions composed of 0.088, 0.880, and 8.795 g/l, respectively, of $AlK(SO_4)_2 \cdot 12H_2O$. From each of these stock solutions a 1.0-mg/l aluminum standard was made up a week later. The standard prepared from the weakest stock solution showed only 0.83 mg/l of aluminum, whereas the strongest stock solution sample checked the regular 1.0-mg/l standard. It is therefore recommended that for acid-reactive aluminum the working standard solution be composed of 0.880 g/l of $AlK(SO_4)_2 \cdot 12H_2O$.

In plant practice, a 50-ml water sample was treated with an excess of 0.2 or 0.3 ml of 0.1 *N* H_2SO_4 beyond the total-alkalinity (methyl orange)

endpoint. The sample was allowed to stand at least 1 hr, and preferably overnight or even longer. This step insured the solution of greater amounts of aluminum before the acetate buffer and aluminon indicator were added. Increased aluminum values were found in Minneapolis water by the acid procedure as opposed to those obtained with the current standard method.

Best results were obtained when the free chlorine in a sample was reduced with sodium thiosulfate just ahead of the buffer treatment in the test. Low readings occurred on testing for aluminum when the water sample was treated with sodium thiosulfate and acid, and then allowed to stand overnight.

Evaluation

It is the opinion of the authors that the present standard method should be abandoned, that all samples and standards used in testing for aluminum should be pretreated with acid, and that the amount of excess acid used per volume of sample or standard appears to make little difference over a wide range of acid concentrations, provided that the final pH of the buffered solutions is rigidly controlled at a fixed value.

The present standard method produces colors which are less intense than are the colors in acid-pretreated standards. Minneapolis aluminum standards prepared without acid pretreatment became unstable progressively on dilution, depending both upon the time and upon the degree of dilution. This effect was attributed in part to an unknown interference in the Minneapolis distilled water. Acid-pretreated standards or samples, however, were found to be consistent and reproducible in color development.

Fluoride Determination

The presence of aluminum has a considerable bearing on the fluoride determination. Various attempts have been made to provide correction curves for fluoride in the presence of aluminum. If aluminum is the only interfering substance in a water supply, correction curves such as those illustrated in Fig. 1 can be constructed for the presence of aluminum, provided that the aluminum is first stabilized by acid pretreatment.

Such correction curves can be applied to the estimation of fluoride and aluminum in Minneapolis drinking water. One of two 100-ml portions is treated with 0.5 mg/l fluoride while the other portion is left untouched. Both portions are then acidified with enough 0.1 *N* H_2SO_4 to neutralize the total alkalinity plus an excess of 0.4 ml of 0.1 *N* H_2SO_4. The samples are allowed to stand for 2 hr, after which the apparent fluoride concentration is estimated by the Megregian-Maier visual procedure (*2*). In a typical case, the portion treated with 0.5 mg/l fluoride showed 0.90 mg/l fluoride while the untreated aliquot showed 0.62 mg/l fluoride. Reference to Fig. 1 gives the aluminum curve which intersects the 0.90 mg/l and 0.62 mg/l ordinates in such a manner that the difference between them is 0.5 mg/l on the true fluoride axis. The graph shows that the unknown amount of fluoride in this case is 1.06 mg/l. Incidentally, the effective aluminum concentration is also obtained at the same time, and, in this example, is seen from the graph to be about half way between *d* mg/l and *e* mg/l. Aluminum concentration curves above *e* mg/l will yield differences greater than 0.5 mg/l fluoride

on the true fluoride axis, while those below *d* mg/l aluminum will yield differences less than 0.5 mg/l fluoride. Aluminum curve, *a*, for example, shows a difference of only 0.3 mg/l fluoride on the true axis. This is known to be incorrect because 0.5 mg/l fluoride was added.

References

1. *Standard Methods for the Examination of Water, Sewage, and Industrial Wastes*, APHA, AWWA & FSIWA, New York (10th ed., 1955). p. 38.
2. MEGREGIAN, S. & MAIER, F. J. Modified Zirconium-Alizarin Reagent for Determination of Fluoride in Water. *Jour.* AWWA, **44**:239 (Mar. 1952).

Discussion

Michael J. Taras

Research San. Chemist, Dept. of Water Supply, Detroit, Mich.

The procedure proposed by Jones and Clark was checked on Detroit water samples along with the standard aluminum procedure. The data are shown in Table 1. Within the limits of experimental error, comparable results were obtained by both procedures on plant water which had passed through the sand filters on the specified days. Appreciably higher aluminum values were demonstrated by the Jones-Clark procedure on the plant water which had been coagulated with filter alum (mixed water), and also on the plant samples collected from the top of the sand filters (applied water).

TABLE 1

Comparison of Aluminum Methods

Time of Sampling	Aluminum Concentration, mg/l	
	Standard Method*	Jones-Clark Procedure
Filtered Water		
10 AM, Sep. 21, 1955	0.27, 0.26	0.25, 0.25
10 AM, Sep. 22, 1955	0.30, 0.30	0.33, 0.30
1 PM, Sep. 22, 1955	0.33, 0.33	0.30, 0.30
3 PM, Sep. 22, 1955	0.32, 0.33	0.30, 0.30
10 AM, Sep. 23, 1955	0.24, 0.26	0.25, 0.25
2 PM, Sep. 26, 1955	0.21, 0.21	0.20, 0.20
Applied Water		
3 PM, Sep. 23, 1955	0.21, 0.23	0.40, 0.40
2 PM, Sep. 26, 1955	0.23, 0.24	0.35, 0.35
Mixed Water		
2 PM, Sep. 26, 1955	0.34, 0.34	0.65, 0.65

* Measurements were made photometrically with a 5-cm light path and a green color filter (525 mμ).

The comparable results obtained with the filtered water indicated the absence of alum floc, a fact confirmed by visual examination of the samples. The higher aluminum values which followed the acidification of the mixed and applied samples, on the other hand, could be ascribed to the solution of a significant amount of alum floc, a finding also substantiated by observation. The standard aluminum procedure responds primarily to such soluble aluminum forms as those yielded by the sulfate salt. Once a floc appears, however, the acid-conditioning procedure recommended by Jones and Clark is useful for redissolving the floc. Thus, the numerical difference found by performing the aluminum determination in the presence and absence of acid provides an estimate of the alum floc in a plant sample. On this account, both the present standard method and the modification proposed by Jones and Clark can serve a purpose in the laboratory control of coagulation.

Separate aluminum standards must be prepared when both the standard and the Jones-Clark procedures are used. The same aluminum standards are not applicable for both procedures.

Filtration

Revision of 'Water Quality and Treatment' Chapter 11

A revision of Chapter 11 of the second edition of Water Quality and Treatment *(American Water Works Assn., New York, 1950), prepared by AWWA Committee 8913 P—Revision of Chapter on Filtration. The members of the committee were R. L. McNamee (Chairman), J. R. Baylis, J. A. Borchardt, K. W. Cosens, Richard Hazen, K. V. Hill, N. J. Howard (deceased), R. C. Kenmir, and W. B. Schworm.*

1. Introduction

George W. Fuller (*1*) divided the development of modern water purification into four periods:

a. Early beginnings (1869–1886)

b. Period of research and development (1887–1902)

c. Period of rapid practical accomplishments (1903–1918)

d. Period of refinements and extensions (1919–1933*)

In 1869 James P. Kirkwood (*2*), chief engineer of the St. Louis water department, published a report which described European water plants and which served as a guide for the design of filters for many years. Much of the early experimental work on slow sand filters was performed at the Lawrence Experiment Station of the Massachusetts State Board of Health, which was put in service in November 1887 and operated under the supervision of Allen Hazen from the summer of 1888 until March 1893. Then, at the turn of the century, Fuller and his associates conducted notable experiments on coagulation and rapid sand filtration at Louisville (*3*).

During the first 15 years of this century, rapid strides in building numerous large filter plants were made. Some of the larger rapid sand filters built during that period were the plants of the Hackensack Water Company and those at Louisville, Cincinnati, Columbus, New Orleans, Toledo, Grand Rapids, Minneapolis, Baltimore, and St. Louis. At the same time, slow sand filters were adopted at New Haven, Denver, Indianapolis, Washington, Philadelphia, Pittsburgh, Toronto, and Wilmington. Modern filtration plants are shown in Fig. 11-1 and 11-2.

The period from 1919 to the present day has essentially been one of refinement of operation, particular attention being paid to the proper conditioning of water for filtration. Water softening, correction of tastes and odors, corrosion prevention, and iron and manganese removal have also been developed to a high degree of refinement during recent years.

2. The Filtration Process

Water filtration may be described as the process by which water is separated from suspended matter by passage through a porous substance. In practice, this porous material is usually a layer of sand. Among sand filters, two general types—slow and rapid—are in common use. Rapid filters, in turn, are also of two types—the widely used free-surface type and the pressure filters.

In slow sand filtration, the water is passed by gravity downward through

* Date of article.

the sand at low velocity, usually at rates ranging from 2 to 10 mgd per acre (0.032–0.160 gpm per square foot). The removal of solid matter is effected by straining through the pores in the surface of the sand layer and by adherence of the solid particles to the sand grains. After an extended period of service, the sand surface becomes clogged and must be cleaned. Water applied to slow sand filters may be pretreated advantageously.

In free-surface rapid sand filtration, the water is passed by gravity downward through the sand at high velocity, usually at rates ranging from 2 to 3 gpm per square foot (125–188 mgd per acre). Pretreatment with a coagulant is essential in order to remove much of the suspended matter by settling. The rapid sand filter is cleaned by a reverse current of wash water which expands and scours the sand, and then carries away the accumulated solids to waste.

The effluent of a rapid sand filter plant is sometimes applied to slow sand filters for "polishing." In the past, this practice was sometimes adopted to insure safe water, but that was before modern chlorination provided a more economical means of achieving the same purpose. Today, double filtration is generally practiced only as a matter of economy in construction or operation, or in order to utilize existing structures.

Pressure filters are enclosed in tight steel cylinders. Cleaning is by backwashing. Rates of filtration range from 2 to 4 gpm per square foot. Pretreatment is always advantageous, and, if the water is for human consumption, pretreatment for pressure filters is as essential as it is for the gravity free-surface filters. Pressure filters are not suitable for large installations because of the limitations in size of units and the impossibility of inspection.

Choice of type of filter must be made on the basis of the character of water to be treated and the overall cost of available systems.

3. The Slow Sand Filter

Slow sand filters were employed extensively in Europe before their introduction in America, and, consequently, the European design was generally adopted for the early American filters. They were first used for fairly clear impounded waters in the northeastern part of the country, where winter conditions dictated covered beds. A cross section of a typical slow sand filter is shown in Fig. 11-3.

Some lake and reservoir waters may be applied to rapid sand filters without pretreatment, but the effectiveness of the coagulation process developed for rapid sand filtration has promoted the use of pretreatment when slow sand filters are used. The coagulation and sedimentation processes used for slow sand filtration are similar to those for rapid filtration, but are usually somewhat more thorough.

Slow sand filtration rates vary widely, depending upon raw-water turbidities, pretreatment, and temperature. Rates usually range from 2 mil gal to 10 mil gal per acre per day. From 200 to 400 mil gal of water can usually be filtered between cleanings.

Uses for which slow sand filters are most successful include the removal of tastes and odors, the bacterial purification of naturally clear waters, the final treatment for highly polluted waters previously treated by rapid sand filters, and the filtration of small supplies where the more skilled supervision required for rapid sand filters is not available.

TABLE 11-1

Sand Characteristics of Slow Sand Filters

Location	Total Area of Filters *acres*	Sand		
		Avg Depth *in.*	Effective Size *mm*	Uniformity Coefficient
Denver, Colo.	10.00	15	0.285	2.79
Indianapolis, Ind.	4.80	15	0.53	2.90
Philadelphia, Pa.				
Torresdale	48.75	26–30	0.28–0.36	1.60–2.00
Upper Roxborough	5.58	36	0.28–0.36	1.60–2.00
Lower Roxborough	2.65	36	0.28–0.36	1.60–2.00
Queen Lane	15.62	28	0.30–0.38	1.50–2.00
Belmont	13.32	36	0.30–0.35	1.60–2.00
Pittsburgh, Pa.	56.00	34	0.29	2.10
Toronto, Ont.	9.60	36	0.22	1.60
Washington, D.C.	29.00	24	0.28	1.60
Yonkers, N.Y.	4.00	36	0.33–0.38	1.90–2.50

Air binding, caused by filling the filter too rapidly or by operating it at too great a loss of head, has given trouble in some slow filter installations. Such binding results in the appearance of "boils" in the sand surface when the filter is drawn down for cleaning.

Filter clogging, which may result from the excessive growth of bacteria within the sand layer as well as from turbidity in the applied water, is much more difficult to correct than air binding. It occurs mostly in warm weather. Very little is known of the basic effectiveness or suitability of chlorine in preventing such clogging, but it appears to be helpful under some conditions and to have no adverse effect on taste and odor. It should be applied with caution, however, until its effect in a given situation is known.

The slow sand filter is usually enclosed in a covered concrete structure. Underdrains laid on the floor are covered with gravel, and over this gravel the sand is placed. Water to be filtered is brought in above the sand, and after filtration it is drawn off through the underdrains.

The initial loss of head through a clean filter is usually less than 1 ft. During operations covering a 50–100-day period, the loss will normally increase to about 4 ft, at which time the filter should be cleaned.

Filter units generally range in size from 0.5 to 1 acre, usually depending somewhat on the total plant capacity. With more units, the flexibility in operation increases, but so does the number of controlling appurtenances.

Sand for slow filters. Where the raw water has not been pretreated with coagulating chemicals, slow sand filters have commonly been used to filter waters of low natural turbidity. The rate of filtration is low—usually less than 3.5 mgd. The sand used is finer than that used in rapid sand filters. Because very large quantities of sand are used in slow sand filters, effort is usually made to utilize local deposits of sand, thus avoiding the cost of freight involved in long-distance shipment.

Reference to Table 11-1 indicates the use of sand sizes of 0.22–0.53 mm in existing slow sand filters. In general, it has been considered that sand

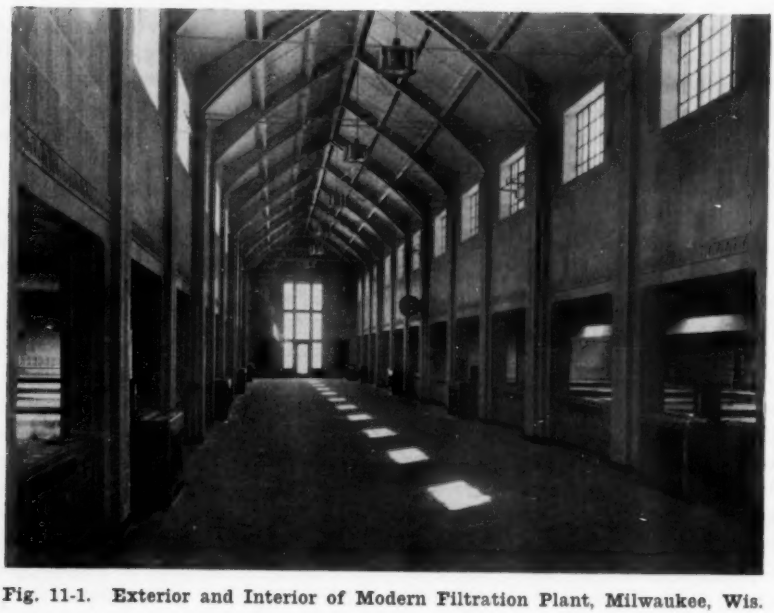

Fig. 11-1. Exterior and Interior of Modern Filtration Plant, Milwaukee, Wis.

with an effective size of about 0.35 mm and a uniformity coefficient of about 1.75 will give satisfactory service, but experience has shown that, with well prepared water, excellent results have been obtained with sand as coarse as 0.50 mm.

As sand offers great resistance to the flow of water, its depth should not be greater than is necessary to provide good purification of the water being filtered. Not only is too great a depth of sand objectionable because of its resistance to flow, but it also involves unnecessary additional cost. Fine sand may be shallower than coarse sand, but the former produces greater losses of head and is likely to clog more quickly than the latter.

The sand depth may be very much less than is traditionally specified. As little as 12–18 in. of graded sand, supported on 12 in. of graded gravel, has been successfully employed, provided that, when cleaned, the original depth of sand is maintained by sand replacement. The gravel layer eventually becomes fouled with a mixture of biologic growth and fine solids, but, if the applied water is well prepared, it is only after many years of operation that the mixture causes interference, either physically or bacteriologically, even with high rates of filtration.

The underdrainage system is usually constructed of full-round or half-round farm drain tile, hollow blocks, or other channeled forms laid with open joints. They are arranged as lateral tributaries to trunks, all leading to one or more outlets.

Several mechanical devices developed for cleaning the sand in slow filters have proved to be reasonably successful. A simple water ejector delivering a mixture of water, sand, and mud to a portable separator is efficient in terms of time and money, but requires much hand labor. Usually about 1 in. of sand is scraped from the surface of a clogged filter and is piled in windrows until approximately 3–4 in. has accumulated, since it is more economical to wash the whole amount at once. Formerly, when deep sand layers were provided, soiled sand was ejected to outside storage until the amount in the filter was reduced to a minimum, and then all of it was replaced at one time. Some plants practice "cultivating" or raking beds between regular cleaning operations. In other plants the units are spaded after scraping. Deep cleaning to the depth of the gravel is necessary after a period of several years.

It is important to maintain a fairly uniform rate of filtration, for which some means of automatic control of the effluent flow is usually provided. Inlet arrangements should insure a fairly constant level of water above the sand and should be so devised as to protect the sand bed from disturbance by the incoming water. In recently built plants, controlling devices, similar to those used for rapid sand filters, are employed.

4. The Rapid Sand Filter

Functional performance. The rapid sand filter differs from the slow sand filter in several respects. Of these, the following have an important effect upon the function and operation of the plant:

a. Relatively coarse sand is used in rapid sand filters.

b. Rapid sand filters are operated at rates which are 15–50 times greater than rates for slow sand filters.

c. Rapid sand filters are cleaned by backwashing instead of by scraping the sand surface.

Coarse sand and high filtration rates can be used with most waters, but only

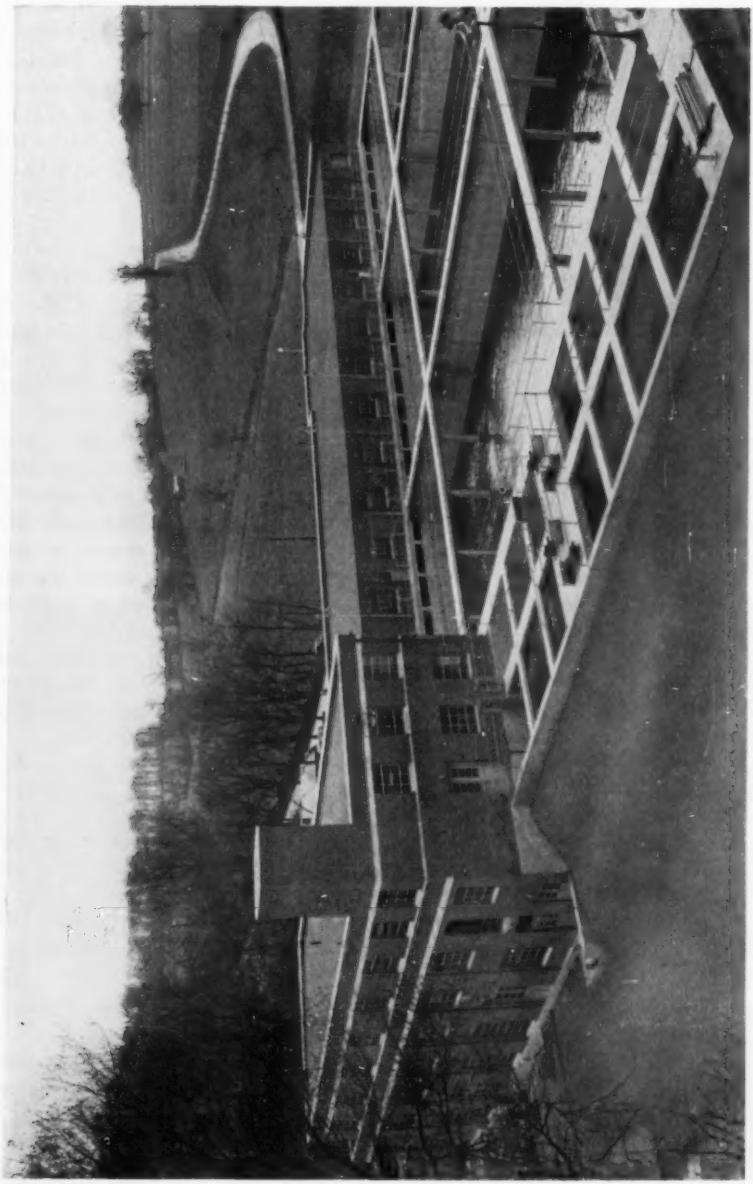

Fig. 11-2. Filter Plant (18-mgd) of the Chester, Pa., Municipal Authority

if some of the suspended matter in the water is removed by pretreatment ahead of the filters. At the same time, backwashing rapid sand filters is a comparatively simple and quick operation and it permits economical cleaning at short intervals.

The conventional rapid sand filter can be used for the clarification of lime-softened water, as well as coagulated water, and the design details of filters for both services are substantially the same. In softening plants, the function of the filter is to remove fine particles of calcium carbonate which have not been removed by the settling basin. The load on filters may be much heavier in softening plants and more frequent backwash is often needed. There is a tendency for calcium carbonate to precipitate on the sand grains and to increase their size. In time, this process may increase the grain size to a point where backwashing is no longer effective and the sand must be replaced. Recarbonation of the water ahead of the filters is frequently employed to prevent this difficulty.

Preliminary treatment. Filtration of raw water through coarse sand at high rates does not remove fine turbidity, colloidal material, or taste- and odor-producing substances and bacteria, as may be accomplished in a slow sand filter. Furthermore, if the water delivered to the filter is turbid, the sand surface is clogged quickly and the filter output is reduced. Therefore, except in unusual circumstances, adequate pretreatment of the water is an essential part of a rapid sand filter plant. (For some industrial purposes, and occasionally in municipal practice, the raw water can be used after simple "straining" through coarse sand. In such cases, pretreatment may not be necessary or warranted.)

Pretreatment, including coagulation with alum or other chemicals, mixing, flocculation, and sedimentation, reduces the suspended matter going to the filters and changes the fine material into a form that is removed by filtration through coarse sand. Settled-water turbidities of less than 10 ppm and sometimes as low as 2–3 ppm are obtained. Pretreatment or conditioning of the water is extremely important because it enables most of the turbidity and bacteria in polluted waters to be removed ahead of the filters. The primary purpose of the filter beds, then, is to remove the suspended coagulated material not removed by sedimentation.

The removal of bacteria by the filters, so vital in the early days of water purification, is now of less importance because of the greater reliance on chlorination. This is especially so at plants with both prechlorination and post-chlorination. Relatively heavy prechlorination is increasingly common for taste and odor control, particularly where the raw water is subject to sewage and industrial-waste pollution. Doses high enough to assure a chlorine residual throughout the settling basin and filters are used, and the water going to the filters is nearly sterile. It must be recognized, however, that some bacteria trapped in floc or in other suspended matter may escape chlorination and must be removed by filtration.

Filtration rates. Assuming adequate pretreatment of the water and reasonable sand size in the filters, the effectiveness of a rapid sand filter is largely a function of filtration rates and the condition of the filter beds.

In the early years of filtration, the capacity of filter plants was based upon operation at a rate of 2 gpm per square foot of filter bed area. This rate was

selected because consistently good water was produced in plants operated at rates not exceeding 2 gpm per square foot. It was recognized, however, that higher rates could be used satisfactorily and most filter plants have been operated at substantially higher rates from time to time in order to meet peak water demands. The rate of 2 gpm per square foot is conservative, and it is rarely necessary to operate a plant at lower rates. On the other hand, if the water coagulates poorly or if a weak floc is produced, operation at higher rates may result in the passage of floc through the filter toward the end of a filter run when the loss of head is high.

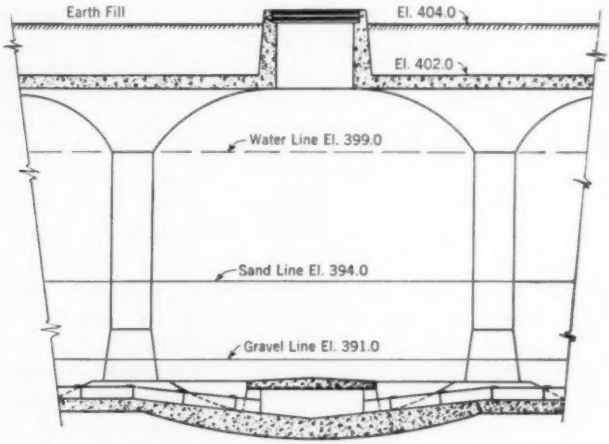

Fig. 11-3. Cross Section of Typical Slow Sand Filter

High-rate filtration. The term "high-rate filtration" is limited to plants where the daily average rate exceeds 2 gpm per square foot or the peak hourly rate exceeds 2.5 gpm per square foot. Under this definition, many of the existing filter plants are operated as high-rate plants at least part of the time. Experiments at Chicago, carried out over a number of years, showed that Lake Michigan water prechlorinated and coagulated with alum (plus activated silica in winter months) can be filtered satisfactorily at rates of 4–5 gpm per square foot. Alum doses of 10–15 ppm were sufficient to produce filtered water of good quality with zero turbidity. The overall bacteriological efficiency was high, but it should be noted that the prechlorine doses were heavy enough to produce a chlorine residual throughout the plant, and the water was practically free from coliform organisms before it reached the filters. Except in the winter, the raw water at Chicago contains a large number of microorganisms, such as diatoms, which strengthen the coagulation. It cannot be assumed that all waters will respond as well. At Detroit and several other cities, however, filters have been operated at rates in excess of 3 gpm per square foot with good results. The relatively coarse and uniform filter sand permits the floc and suspended

matter to penetrate several inches into the filter. If this penetration did not occur, the surface of the sand would clog quickly and long filter runs would be impossible. The aim is to obtain a combination of pretreatment and filtration that will permit the maximum flow of water but will not result in suspended matter passing through the filter.

gallery design and installation of dehumidification equipment.

Difficulties with filter beds include:

a. Gradual clogging of the sand as a result of excessive loading of the filters, leading to incomplete removal of floc from the top of the beds during backwash

b. Contraction of the bed and pulling away of the sand from the walls

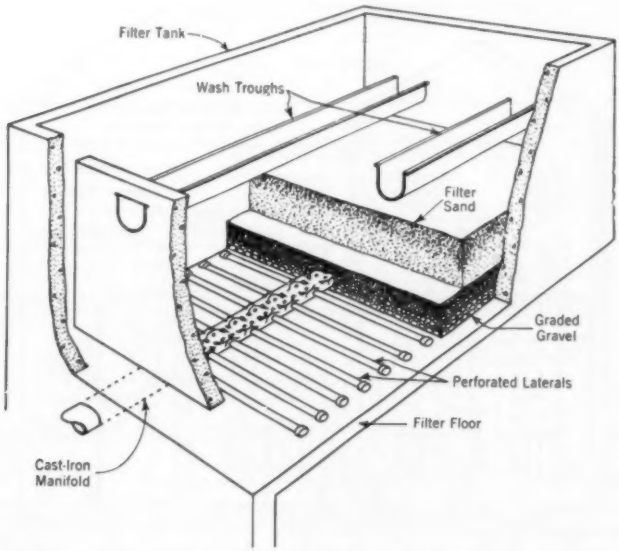

Fig. 11-4. Cutaway View of Typical Rapid Sand Filter

Operating difficulties. Difficulties in the operation of rapid sand filters are of two kinds: [1] failure of mechanical equipment and controls installed to regulate the flow of water, and [2] failure of the filter bed itself.

Control equipment now available is more reliable than the early types and, if properly installed, should give little trouble. To facilitate keeping the controls in good order, much attention and expense have been devoted to the improvement of such factors as filter

of the filter, when the sand has been allowed to become dirty

c. Binding together of the sand by sticky flocculated material to form mudballs throughout the filter

d. Movement of gravel layers and perhaps mixing of the sand and gravel, caused by uneven backwash of filters; this results in irregular thickness of the sand bed and, possibly, if allowed to continue, in the escape of sand through the gravel to the underdrains and clear well.

Most of the above difficulties can be avoided by making use of the following measures:

a. Underdrain systems designed to prevent excessive backwash rates and to assure even distribution of water over the entire bed

b. Gravel which is large enough and suitably graded to resist movement during the backwash

c. High-velocity backwash, preferably accompanied by a surface wash

d. Washing filters frequently enough to prevent the accumulation of dirt on the surface of the filter and the formation of mudballs

e. Prechlorination to keep the sand clean and free from sticky substances likely to form mudballs.

5. Arrangement and Unit Capacities

Functional arrangement. The filtering units in a rapid sand filtration plant are usually placed in a compact group as close to the sedimentation basins as local considerations will permit. Such an arrangement produces minimum head losses and results in the most economical design. In many modern rapid sand filtration plants the various filter units are arranged on one or both sides of a pipe gallery. All pipes or other conduits for carrying water, and other appurtenances for control of the filter units, are placed within this gallery. Details of rapid sand filter plants are shown in Fig. 11-4–11-6B.

Other groupings of filter units are possible. The most important factors

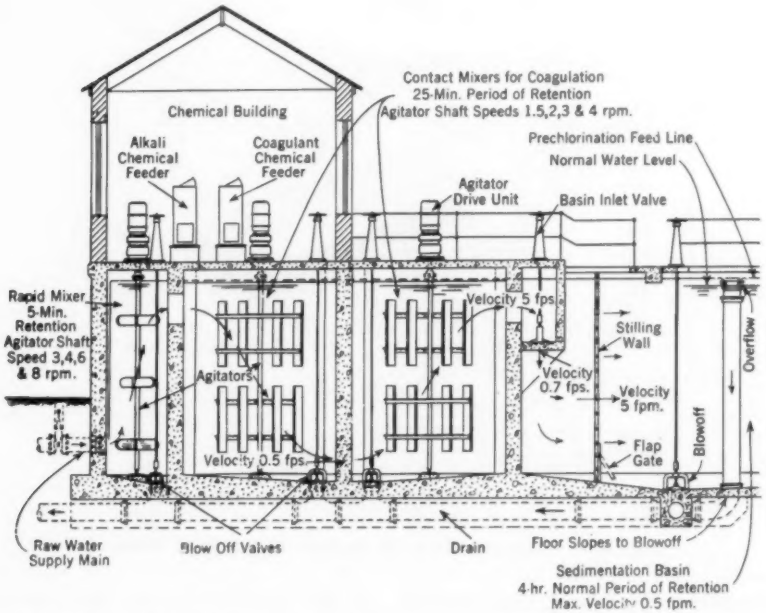

Fig. 11-5A. Flow Diagram of Typical Rapid Sand Filter Plant (Left Half)

in making the final decision on filter arrangement are convenience to the operator, economy of construction, and provision for future expansion.

The design of the pipe gallery is usually complicated by the amount and size of the equipment which must be placed within this space. Details of a modern pipe gallery are shown in Fig. 11-7. From the aspect of the economics involved, the gallery should be as small as practical. Any waste space in this area must necessarily increase the width of the operating floor and the volume of the filter building. Although economy is a factor, adequate space must be provided in the gallery for convenience of inspection and for removal of faulty equipment. Manholes or points of access should be provided for easy handling of heavy pieces of equipment. Large plants frequently provide a hoist at each of these points, whereas the smaller plants may require only the provision of a hook set into the structure for supporting a block and tackle rigging.

Structurally, the filter gallery and the operating floor above it should be designed to withstand not only the weight of the equipment in place, but also the weight of this equipment concentrated on the floor before and during erection.

The pipe gallery should be arranged to cope with the condensation of moisture. A few plants have insulated the piping. Other plants paint the piping and rely on ventilation of the gallery for humidity control. Ventilation

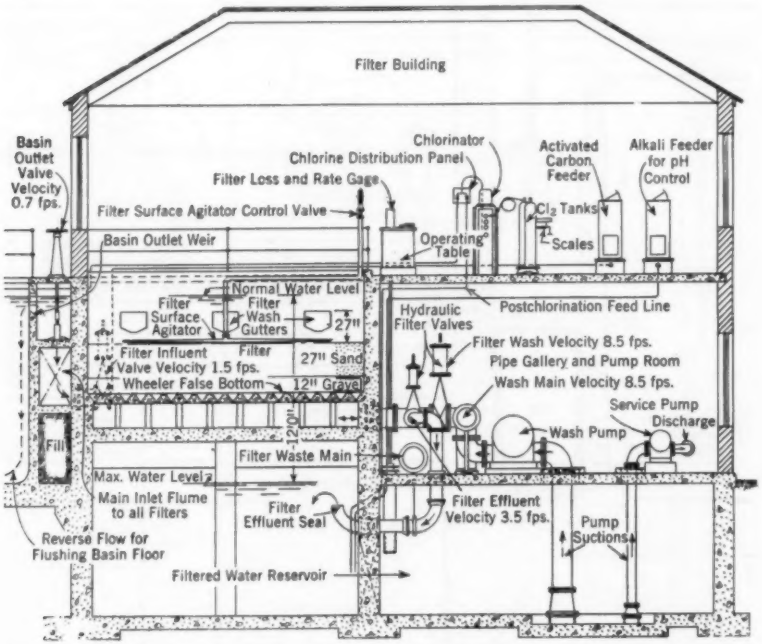

Fig. 11-5B. Flow Diagram of Typical Rapid Sand Filter Plant (Right Half)

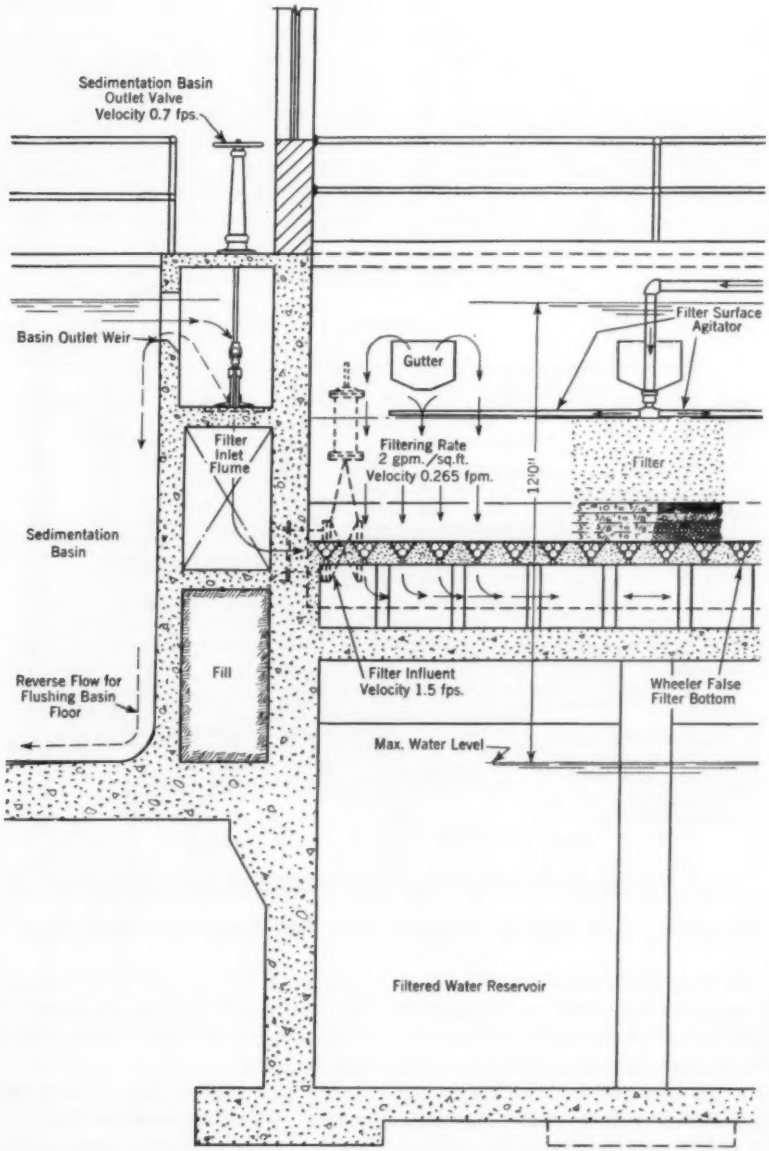

Fig. 11-6A. Cross Section of Typical Gravity Filter Unit (Left Half)

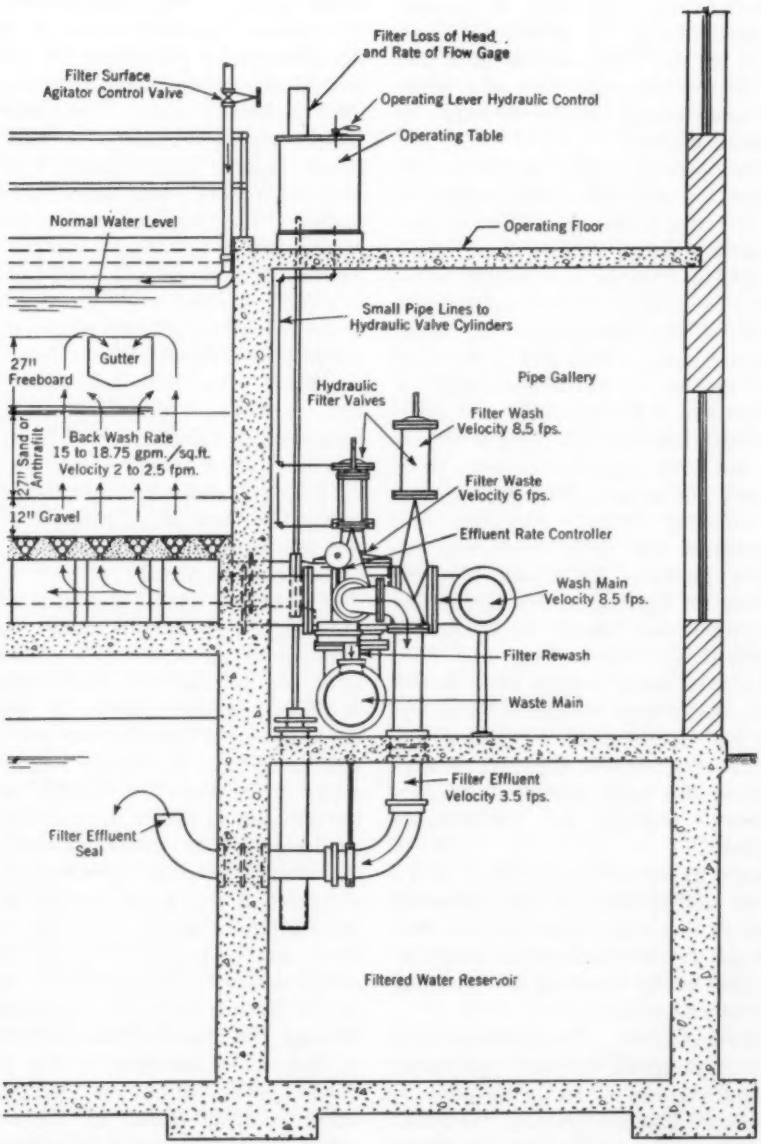
Filter Loss of Head and Rate of Flow Gage
Filter Surface Agitator Control Valve
Operating Lever Hydraulic Control
Operating Table
Normal Water Level
Operating Floor
Small Pipe Lines to Hydraulic Valve Cylinders
27" Freeboard
Gutter
Pipe Gallery
Hydraulic Filter Valves
Filter Wash Velocity 8.5 fps.
27" Sand or Anthrafilt
Back Wash Rate 15 to 18.75 gpm./sq.ft. Velocity 2 to 2.5 fpm.
Filter Waste Velocity 6 fps.
12" Gravel
Effluent Rate Controller
Wash Main Velocity 8.5 fps.
Filter Rewash
Waste Main
Filter Effluent Velocity 3.5 fps.
Filter Effluent Seal
Filtered Water Reservoir

Fig. 11-6B. Cross Section of Typical Gravity Filter Unit (Right Half)

solely through the base of the operating table is rarely sufficient to prevent the accumulation of moisture. Floor drains in this area are essential and may mean the employment of a sump and sump pump for the discharge of collected drainage.

The operating floor should be arranged for maximum convenience of the operating personnel. Controls and indicating gages which have been brought up from the pipe gallery below should be installed near the appropriate filter unit on floor stands or on an operating table. This area is the focal point for visitors to the plant and, for this reason, it is frequently well decorated and finished. Lighting is important not only from the visitors' point of view but as an aid to the operator in cleaning filters. Skylights and clerestories are quite commonly included, therefore, as a means of enhancing the lighting in this area. The open filter units should be adequately protected by hand railings and floor curbs. All parts of each filter should be accessible from walkways for necessary bed maintenance. Lighting fixtures over filters and skylights or windows in this area should all be conveniently reached for maintenance purposes.

Some thought, too, should be given to the arrangement of the operating floor, to the maintenance of the filter bed, and to such facilities as might be required in the handling and cleaning of sand and gravel.

Unit capacities. The filter box itself may be designed for any convenient nominal capacity. In the medium- or large-sized plants, especially, it is rather general practice to construct the filters in some even multiple or fraction of 1-mgd capacity. In the very small plants the bed size becomes subordinate to flexibility, which may result in considerable deviation from average practice. The present tendency seems to lie in the direction of increasing the size of individual filter units and reducing the total number of units. This results in some economy because of a reduction in the required total number of valves, rate controllers, and other appurtenances. The largest units presently constructed are in the 5–6-mgd size. The upper limit of unit filter capacity is usually reached when the flexibility of the overall installation is affected or when backwashing facilities become a problem. The latter condition has been overcome in some installations by backwashing a single large filter in two sections, which may be accomplished by placing a gullet through the center of the filter bed and dividing the bottom into two parts, each of which may be separately washed.

Hydraulics. Rapid sand filters may be designed to operate under conditions where the water level within the filter box is essentially held constant, is allowed to vary slightly, or rises as the head loss in the bed develops. The last condition produces wide variations in the water level within the filter box and is rather infrequently encountered. The water elevation on the filter is usually controlled by the elevation of the water surface in the sedimentation basins. Rising water levels can be produced in the clarification basins by reducing filter capacity, or by increasing total head losses through the filters without adjustment in the raw-water flow to the plant. Under such conditions, coagulated water is being stored by the simple expedient of allowing it to accumulate in the channels and basins ahead of the filter and at the expense of the freeboard in these units.

In contrast to conditions that create rising water levels on the filters, some water treatment plants are equipped to permit the water surface in pretreatment basins to drop below the normal operating levels. This is accomplished by means of orifice openings into the launders or valved outlets from the basins, permitting a continuous rate of filtration to proceed using the storage capacity of the sedimentation basins above these takeoff points during short-time shutdowns of the raw-water supply. During such shutdown the filters are operating under conditions where the water level on the bed is continuously falling.

Changes in water elevation at the filter may be controlled within limits by float-controlled valves, or by master controllers placed on the filter gallery influent and connected so as to change simultaneously the rate controller settings of the individual filters if the influent water elevation varies beyond predetermined limits.

Once the allowable range of elevations for the water surface on the filters is selected and the hydraulic losses are computed back to the pretreatment units, then the basin elevations and related filter box elevations may be determined. Freeboard is always a necessary consideration in designating filter box dimensions.

The head loss through a clean filter normally varies from 6 in. to 12 in. of water. As the floc accumulates within the sand bed, this head loss increases to values of 6–10 ft of water. The rate of filtration is kept constant by a continual automatic adjustment of the rate controller so as to maintain a constant total head loss through the filter and rate controller. The point of backwashing is usually not determined by the maximum limit of adjustment of the rate controller but by the turbidity of the filter effluent. An increase in this turbidity reflects the passage of fine material by the filter. Fine particulate matter is carried through the sand bed when the interstices of the bed become sufficiently clogged with entrained floc, resulting in the increase of local velocities to a level beyond which agglomeration and retention of fine particles in the sand bed are impossible. In some plants, the head loss through the sand bed may be as low as 1.5 ft when backwashing is considered necessary.

During the process of filtration the head loss which develops in the sand bed frequently exceeds the depth of water available above the sand for maintaining flow. The difference in these values is termed negative head and may be thought of as a pull or suction on the water column passing through the bed. This suction results from the fact that the effluent pipe is located below the sand bed. Whenever the rate controller calls upon this reserve head for maintenance of flow, the pressure at the sand-gravel interface is reduced below atmospheric pressure by the amount of the negative head required. Water plant operators frequently find that under conditions of reduced pressure, dissolved gases are released from the water being filtered. These gases collect in the gravel, causing air binding of the filter bed, and they may result in extensive damage to the gravel layers at the time of the next backwash. The design engineer can avoid all such difficulties by making the filter box sufficiently deep so that negative heads are seldom necessary to maintain flow. This is not the entire answer to the problem, however, because head limitations or economic factors frequently make such filter

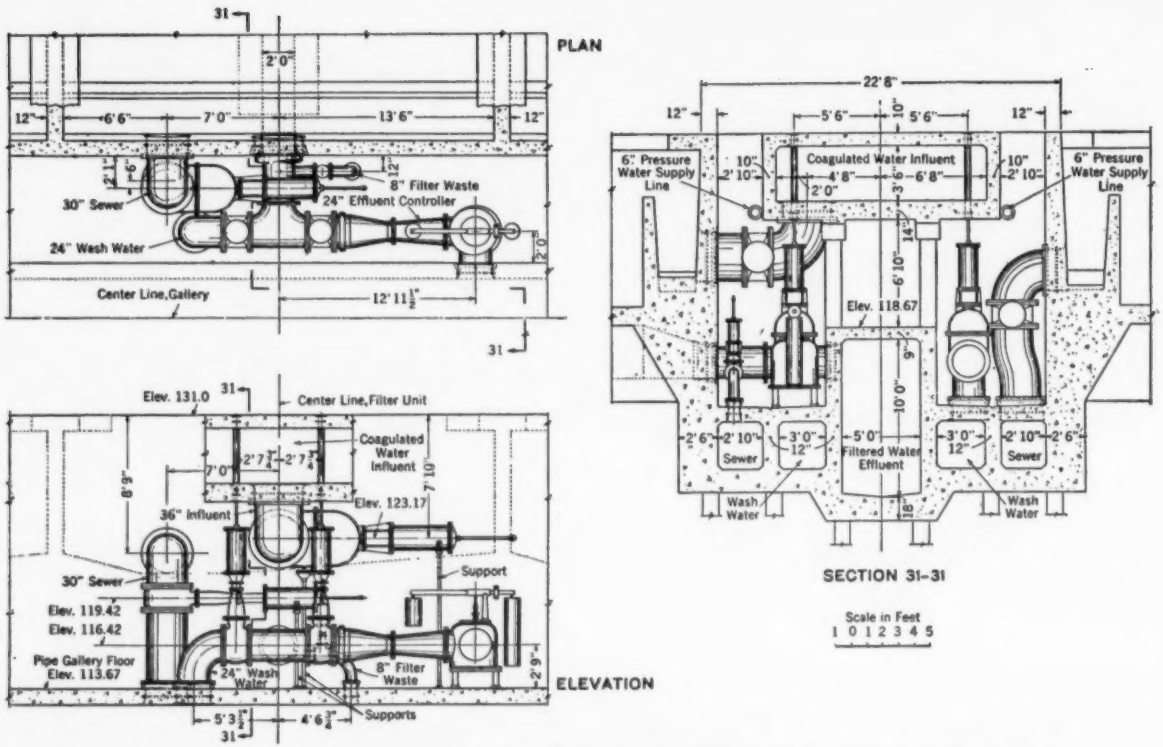

Fig. 11-7. Details of Filtration Plant Pipe Gallery, Springwells Filtration Plant, Detroit, Mich.

depths impossible. Where negative head occurs, the responsibility for proper filter functioning rests on the shoulders of the filter attendant.

6. Filtering Media

Filtering media are defined as the material (usually natural or prepared sand) used to separate coagulants, turbidity, color, and bacteria from water by straining action.

The gravel sometimes used in supporting the sand in a filter tank is usually thought of as one of the filtering media, although the gravel has an entirely different function. The gravel supports the sand and assists in the distribution of the backwash water. With the porous-plate type of underdrain, the gravel may be omitted.

Filtering materials. Sand has been the most universally used material for filtering, but, in recent years, mechanically crushed quartz, anthracite coal, and other materials such as calcined diatomaceous earth have been advocated as substitutes for sand.

Gravel, either natural or crushed, has usually been used to support the sand, although in some filters crushed anthracite has been employed to support the finer upper layers of anthracite.

Sand for rapid filters. The AWWA specification for filtering materials (*4*) states that:

> The grade of sand to be used in any particular filter cannot be specified without consideration of local conditions and controlling factors. For purposes of definition in these specifications, the following limits have been established for the 10 per cent, or effective size: fine sand, 0.35 to 0.45 mm; medium sand, 0.45 to 0.55 mm; coarse sand, 0.55 mm or larger. . . . In general, coarse sand will permit longer runs between washings than fine sand, and with good pretreatment facilities and close technical control will yield results adequate for most purposes. Coarse sand is less effective than fine in the removal of turbidity and bacteria.
>
> *Coarse sand* is applicable when: [1] pretreatment is consistently good; [2] the highest degree of filtration is not required; [3] benefits from longer filter runs and less wash water offset any disadvantages from lower water quality; [4] filter design will permit the necessary high backwash rates.
>
> *Fine sand* is applicable when: [1] pretreatment is sometimes poor; [2] the maximum removal of turbidity is needed; [3] the maximum removal of bacteria is needed to safeguard health; [4] benefits of longer filter runs and lower wash water consumption are not of great importance; [5] backwash is limited by filter plant design to rates sufficient for cleaning fine sand only; [6] rapid buildup of grain size with calcium carbonate following softening is expected.
>
> *Medium sand* is a compromise between the coarse and fine and is frequently suitable under average conditions.

After numerous experiments on filter sands at Baltimore, James W. Armstrong (*5*) arrived at the following conclusions regarding the selection of sands for rapid filters:

> *From the standpoint of filtration,* it is desirable to have a sand that will (*a*) prevent any floc passing through the filters, (*b*) hold floc as loosely as possible in order to permit easy washing and prevent the formation of mud deposits, (*c*) hold as large a volume of floc as possible without clogging.
>
> *From the standpoint of washing,* it is desirable to have a sand of such size that it will (*d*) cleanse itself and be free from adhering floc at the end of a wash, (*e*) permit the passage of water at sufficient velocity to remove all the sediment without losing sand.

A board of state sanitary engineers in the Great Lakes and Upper Missis-

TABLE 11-2

Sand Characteristics of Rapid Sand Filters

Location	Year Built	Sand		
		Avg Depth *in.*	Effective Size *mm*	Uniformity Coefficient
Columbus, Ohio	1923	24	0.55	1.60
Baltimore, Md.	1928	24	0.60	1.20
Washington, D.C. (Dalecarlia)	1928	20	0.48	1.40
Philadelphia, Pa.	1929	28	0.40–0.55	1.50–1.70
St. Louis, Mo. (Howard Bench)	1929	24	0.46	1.48
Detroit, Mich. (Springwells)	1931	20	0.54	1.52
Fort Wayne, Ind.	1933	24	0.45	1.20
Denver, Colo. (Moffat)	1937	27	0.48	1.41
Cincinnati, Ohio	1938	27	0.44	1.50
Toledo, Ohio	1939	29	0.35–0.45	1.65
Milwaukee, Wis.	1939	27	0.55	1.20
Grand Rapids, Mich.	1940	30	0.50	1.33
Indianapolis, Ind. (Fall Creek)	1941	27	0.39	1.40
Chicago, Ill. (South District)	1942	24.5	0.62–0.70	1.30–1.50
Moline, Ill.	1948	24	0.50–0.55	1.65

sippi basins suggest a sand depth of 24–30 in., a sand with effective size of 0.35–0.80 mm, and a uniformity coefficient of not more than 1.7 (*6*).

Table 11-2 gives a representative indication of current practice in the selection of sand for rapid sand filters. It will be noted that the sands are somewhat coarser than those used in the slow sand filters shown in Table 11-1.

The present trend is toward better preliminary treatment of the applied water, coarser sand, and higher rates of backwashing, with surface wash as an essential part of the installation.

Filter sand to be used where aggressive or low-pH waters are to be filtered should be specified to be composed of hard, durable grains, free of clay, loam, dirt, and organic matter. Not more than 5 per cent of its original dry weight should be lost after immersion for 24 hr in warm concentrated hydrochloric acid. Where filters follow lime-softening treatment or are to be used to filter alkaline waters, acid solubility of the sand is of minor importance.

The AWWA specification for filtering materials (*4*) includes alternate methods of defining sand size and selecting a sand. The first method assigns the limiting of sizes to stated percentages by weight, stating, for example, that "the 10 per cent size shall be between 0.35 and 0.40 mm."

The second method of classification defines the percentage of sand that shall be finer than stated grain sizes. The specification states, for example: "The percentage of sand finer than 0.4 mm shall be between — per cent and — per cent of the total." By fixing percentages that correspond to the separation sizes of standard sieves, the results of a sieve analysis can be used directly without plotting. Typical specifications for fine, medium, and coarse sands are given in Table 11-3.

A well graded sand which is approximately of the desired effective size will be satisfactory if [1] the 1 per

cent size is not less than 0.5 times the 10 per cent size, [2] the 60 per cent size does not exceed 1.7 times the 10 per cent size, and [3] the 99 per cent size does not exceed 2 mm or 4 times the 10 per cent size, whichever is smaller.

The grading of the three sands, specified according to sieve sizes, is given in Table 11-4.

Anthracite for rapid filters. The use of anthracite coal as a filter medium is steadily increasing. At the present time it is being used in municipal water treatment plants in 23 states. Its use goes back to 1914, when it was installed in the filters at Cumberland, Md.

Anthracite coal has an average specific gravity of 1.5, as against 2.65 for silica sand and crushed quartz. The weights per cubic foot are approximately 55 lb for coal and 200 lb for sand. Thus, a ton of anthracite coal will fill almost twice as much filter volume as a ton of sand. This tends to compensate for its considerably greater cost.

The principal advantages claimed for anthracite material are: [1] because of its lower specific gravity, only about half the backwashing velocity needed for sand is required for anthracite material of the same size to produce equal expansion, and [2] anthracite material of the same effective size as sand has a greater porosity.

Anthracite filters are used successfully at Washington, D.C., Wheeling, W.Va., Rock Island, Ill., and other places.

7. Underdrainage Systems

Underdrains for rapid sand filters serve a dual purpose: they provide an outlet for the water passing through the filter, and are the means for supplying wash water to the underside of the filter medium. Because wash water is applied at rates many times greater than filtering rate, the design of filter underdrains is governed primarily by the necessity of delivering wash water evenly to the entire area. If the wash water is applied unevenly, the sand will shift in the filter, possibly mixing the sand and gravel layers.

Filter bottoms are of the manifold and perforated-lateral type or of the false-bottom type. Where the manifold and perforated-lateral system is used, an even distribution of wash water is obtained by keeping the head loss high through the orifices in the laterals, as compared to the friction losses in the manifold and laterals. Where wash water is distributed by means of a false bottom, even distribution can be obtained throughout the filter with much lower friction losses through the orifices. Good results have been obtained with both types of filters. The false-bottom arrangement permits high rates of backwash with considerably lower operating pressures. The manifold and perforated-lateral system has been popular for many years, but in recent installations there has been a pronounced trend toward the employment of false bottoms, porous plates, and other patented arrangements. The perforated-lateral underdrain system comprises a grid of manifold, headers, and laterals, covering the entire filter bottom. The sizes of the various pipes are selected to hold head loss through the underdrain system to reasonable limits. The underdrain system may be of cast iron, wrought iron, asbestos-cement, or concrete. In some small plants, or plants handling unusually corrosive water, copper and stainless-steel piping has been used. Cast-iron pipe has been used most frequently, sometimes with cement-mortar lining

TABLE 11-3

Grain Size Distribution by Per Cent Size

Per Cent Size*	Grain Size—*mm*					
	Fine		Medium		Coarse	
	Min.	Max.	Min.	Max.	Min.	Max.
1	0.26	0.32	0.34	0.39	0.41	0.45
10†	0.35	0.45	0.45	0.55	0.55	0.65
60†	0.53	0.75	0.68	0.91	0.83	1.08
99	0.93	1.50	1.19	1.80	1.46	2.00

* If the size distribution of the sand particles is such that 1 per cent of a sample is finer than 0.34 mm, the sand is considered to have a 1 per cent size of 0.34 mm.
† The ratio of the 60 per cent size to the 10 per cent size shall not exceed 1:70.

as protection against corrosive water. Failure or clogging of the underdrain system leads to improper backwash and loss of sand, and complete rebuilding of the filter is usually required. Therefore, only durable material should be used in underdrain systems. One patented filter bottom consists of hollow glazed tiles placed so as to provide waterways in the top of the tile, similar to laterals and perforations, for distributing water throughout the filter.

False bottoms are usually installed to provide a water passageway 6–12 in. in height under the filter. In the simplest type, concrete or steel bottoms contain one $\frac{3}{8}$–$\frac{1}{4}$-in. orifice for each 1-ft length of filter bottom. The size and spacing of the orifices depend upon the head loss to be provided. High-velocity discharges upward from the orifices into the gravel are prevented by installing hoods over the openings or by some other means of laterally deflecting the water. False bottoms consisting of slats or steel gratings have not been successful because of uneven backwashing at high rates.

The Wheeler bottom is constructed in the form of inverted truncated pyramids, with the water connection at the apex of the pyramid and the pockets filled with glazed earthenware spheres of selected size. Sectional cast-iron forms can be rented by contractors for pouring the concrete pyramids to standard dimensions.

TABLE 11-4

Grain Size Distribution by Sieve Size

Sieve Number		ASA Sieve Opening *mm*	Effective Size—*per cent passing sieve*		
Tyler No.	US No.		Fine 0.35–0.45 mm	Medium 0.45–0.55 mm	Coarse 0.55–0.65 mm
14	16	1.19	94–100	84–99	68–93
20	20	0.84	71–97	49–84	30–71
28	30	0.59	31–73	14–39	6–31
35	40	0.42	6–25	2–6	0–1
48	50	0.30	0–3	0–1	0

Porous plates of vitrified crystalline aluminum, more commonly known as corundum, have been used successfully in many filter plants. The plates are supported at a height of several inches above the filter floor by means of stainless-steel bolts or concrete ridges, thus providing the necessary channels for distributing water throughout the filter. Where porous plates are used, the sand is placed directly on top of the plates without a gravel layer. This effects some economy of construction by permitting the use of shallower filter boxes. It also eliminates difficulties resulting from the disturbance of the fine gravel layer and mixing with the sand. The water passages in the porous plates are small and some clogging takes place, but this has caused difficulty only where the filters are used in connection with a lime softening plant or where the water has not been properly coagulated.

8. Washing Systems

Backwash water should be applied to the underdrainage system at a rate sufficient to remove from the surface and within the sand bed all material filtered out during the preceding filter run. The rate of application depends upon the character of the material filtered out, the temperature of the water, and the size and specific gravity of the filter media. For a given rate of application, the bed expansion is greater for fine than for coarse sand and increases with a decrease in the temperature of the water. Under average conditions, a rate of application which produces a bed expansion of 50 per cent gives satisfactory results, especially if a surface wash has been used at the beginning of the wash. As shown by Hulbert and Herring (*7*) in their investigations made at Detroit, it is necessary to use very high backwash rates to produce a 50 per cent expansion if the sand is relatively coarse, especially when the water reaches temperatures prevailing during the summer months. The effect of temperature upon the backwash rate is indicated by Table 11-5 (*7*). Experience has shown that with relatively coarse sand, satisfactory cleansing can be accomplished under most conditions even though the bed expansion is somewhat less than 50 per cent, provided a surface wash is used at the beginning of the wash.

Present design practice usually provides for a normal wash rate of 24–30-ipm rise and a maximum rate of 30–36-ipm rise.

The length of the filter wash depends upon the nature of the matter being removed by the filter, the size of the sand grains, the water temperature, and, especially, whether or not a surface wash system is used. Under average conditions, however, a wash lasting 3–5 min is usually sufficient.

If the water being filtered contains considerable turbidity or is possibly contaminated, it is advisable to allow a few minutes' waste of the filter effluent after washing the filter or to let the filter stand for a few hours before returning it to service. In most plants with adequate pretreatment facilities, the filters are returned to service immediately after being washed.

Before the advent of the surface wash, even though fairly high backwash rates were employed, many of the older plants had great difficulty in achieving a clean filter. Upon expansion of badly clogged beds, particles of sand held together with mud would form balls which could not be broken by the backwash alone.

An attempt to improve washing conditions was the so-called air wash, by

which air is introduced along with the backwash water to increase agitation of the sand. Although improvements of this method have been reported, the air wash has not proved entirely satisfactory for municipal use in this country. The air wash requires a specially designed underdrainage system and increases the possibility of mixing the smaller-size gravel with the sand. In recent years, designers have favored the surface wash as an adjunct to the regular backwash because it is safer to use and is generally considered more effective than the air wash.

The surface wash is applied through high-velocity horizontal jets of water introduced slightly below the surface of the expanded bed. These jets create an extreme turbulence that breaks up mudballs and thoroughly scours the sand particles. At the beginning of the wash, both the surface wash and backwash are started at about the same time, with the backwash rate set so as to produce a bed expansion of about 10 per cent. The rate at which the surface wash is applied varies widely for different types. The fixed type requires 4–8 gpm per square foot, depending upon the type of water, the effectiveness of the pretreatment, the design of the particular system, and the pressure of the available water supply. The revolving type, usually referred to as the Palmer system, requires only 0.5–1 gram per square foot of filter. The surface wash is generally used over a period of 1–3 min at the beginning of the wash. After the surface wash is stopped, the backwash is continued for 2 or 3 min, but at a much higher rate.

The Baylis fixed-type surface wash system (*9*) is constructed with horizontal pipes set above the water surface at a spacing of 24–36 in. These pipes supply vertical pipes (constructed preferably of red brass) set at about the same spacing as the horizontal pipes and terminating in perforated brass caps set just above the sand bed. Each brass cap is drilled through the sides with four to eight holes. Other fixed piping arrangements provide for setting the horizontal distributing pipes just above the sand bed, with horizontal holes drilled through both sides at spacings of 6–8 in. The diameter of the holes is designed for the desired flow and the available water pressures. The fixed type of surface wash may be used with as low as 20 ft of head at the jets, although 50–60 ft is desirable where there is a tendency for mudballs to form. The Baylis surface wash is

TABLE 11-5

Rate of Application of Wash Water Required for 50 Per Cent Expansion of Bed

30 Per Cent Size*—*mm*	Rate of Rise of Wash Water—*ipm*	
	32°F	70°F
0.40	12.3	20.2
0.50	18.5	29.2
0.60	25.3	39.0
0.70	32.7	49.3
0.80	41.3	60.3

* This size appears to have the greatest significance in correlation with the performance of a given sand under the washing condition.

preferable to the other type because if the holes become enlarged the caps may be replaced at small expense (*10*).

The rotating or Palmer surface-washing system consists of one or more units, each of which includes two horizontal arms supported through a swivel connection to a wash water trough. Horizontal jets propel the arms in a manner similar to a rotating-type lawn sprinkler. Although the Palmer system uses only 0.5–1 in. gpm per square foot of filter the manufacturers recommend that a pressure of 50–75 psi be provided at the jets.

The cleaner beds produced by the surface wash permit longer filter runs and result in using less wash water. A further saving in wash water is effected by reason of the shorter washing periods possible when the surface wash is used.

A rate-of-flow controller or other flow-limiting device should be provided for the backwash. Rate-of-flow controllers with indicating and recording equipment are frequently installed for both the backwash and surface wash, and these provide permanent records of all washing operations.

Wash water supply. Water for washing filters may be supplied either from a tank or from one or more pumps taking suction from the clear well and discharging directly into the washing system. The former is generally used in plants having large filter units and the latter is commonly used in plants having smaller filter units. Pumps have been used with satisfactory results in some medium-sized plants with capacities up to 25 mgd. Where there is a high maximum demand charge for power and backwashing cannot be done during off-peak high-service pumping periods, the operating expense with water pumps becomes rather high. Some small plants are supplied with backwash water from the distribution system, but this is poor practice because such an arrangement may produce sudden drops in the system pressure. The system pressure is usually much higher than necessary for backwashing. The use of such high-pressure water represents a wasteful use of power and unless proper protective devices are provided, there is always the possibility that a filter bed will be overturned because of the pressure.

A wide variation in wash water tank storage capacity is provided in various plants. The average storage capacity appears to be about 70,000 gal for every 360 sq ft of filter area. The storage requirements should be analyzed for each plant, taking into consideration the most adverse conditions which may be expected. In any case, however, storage capacity equivalent to that required for at least two complete filter washes should be provided.

Wash water tanks should be high enough to provide the head required to deliver the desired flows to the most remote filter units. In actual practice, heads of only 35 ft or less between the bottom of the tank and the top of the filter sand are provided in most plants, although sometimes there is 50 ft.

Steel standpipes usually are less expensive than other types of tanks and are ideal for wash water storage, particularly in milder climates where freezing is not a serious problem. If standpipes are built to provide 50–60 ft of head when full, the head is sufficient for both the surface wash (of the fixed type) and the backwash. The wash water tank should be so located that the length of run to the filter pipe gallery is kept to a minimum.

It is preferable to arrange the piping so that the pump will discharge into

the pipe leading to the tank at some point between the wash water meter and the tank, thus insuring that all water will be metered even though the pumps are running during washing operations. The pumps should be set so as to insure that the suctions will be flooded at all times.

Horizontal, centrifugal, double-suction pumps are widely used, although those of the horizontal, centrifugal, side-suction type are also common. In general, the double-suction pump is more efficient and requires less maintenance, but it has a higher initial cost than the side-suction type.

Where the pumps can be located over the effluent conduit or over the clear well, it is often economical to use vertical pumps of either the deep well turbine type or the mixed-flow type. Vertical centrifugal pumps with the pumps set in a dry well and the motors supported on the operating floor above are sometimes used, but, because of the tendency of the bearings to give trouble, such pumps are not as trouble free as the types mentioned above.

When pumps are used for supplying the backwash water, the surface wash should be supplied from some other source which will provide a higher pressure, such as a main in the distribution system, or from a separate pump designed for the higher pressure needed for the surface wash. Mixed-flow pumps should not be used for supplying wash water directly to the filters because this type of pump tends to overload the motor when the discharge is throttled.

In order to prevent damage to the pumps when they are operated for a considerable time against a closed discharge, a relief line $\frac{3}{4}$–1 in. in diameter, depending upon the pump capacities, should be provided. This line should be without valves and should be installed between the discharge header and the clear well or at another convenient point of disposal.

If it is possible for the level in the clearwell to be drawn down below the pump suctions, a float switch should be provided for stopping the pump when the water level in the clearwell drops below a safe level.

Each pump discharge should be equipped with a check valve of proved nonslam characteristics.

Piping. The filter gallery piping is one of the most important components of a filter plant. The designer should use his ingenuity to develop an arrangement of piping which will satisfy all functional requirements and insure ease of maintenance and operation. One should be able to walk the length of the pipe gallery without having to climb over piping and without walking through puddles of water; it should be possible to remove any individual valve without the disassembly of any large amount of piping. It may therefore be necessary to set some of the valves in positions that are not ideal from the standpoint of minimum wear, but this is considered preferable to an arrangement which might otherwise be extremely difficult to maintain.

Filter piping, including concrete conduits, should be designed with the utmost care to insure adequate capacity, tight joints, and ease of maintenance. When designed for the following maximum velocities, the head losses are kept reasonably low and without excessive cost: influent and effluent manifolds or conduits, and connections to filters, 2–3 fps; backwash and surface wash piping, 10 fps; and waste wash water, 3–5 fps.

In large plants, the influent, effluent, and waste wash water manifolds usually are constructed of rectangular concrete flumes or conduits. Similar construction is often used in small plants for the influent manifold and waste wash water conduit. Such conduits must be constructed with contraction joints provided with copper or rubber waterstops and designed with special emphasis on watertightness. Large conduits should be provided with means of access for purposes of maintenance. In older plants it was customary to use cast-iron pipe and fittings for all metal piping except small auxiliary lines, and, although this practice still is common, there has been a growing trend toward the use of welded steel pipe and fittings for piping larger than 12 in. Steel plate not less than $\frac{1}{4}$ in. in thickness should be used for such work, and the interior of the pipe should be lined with hot-applied coal-tar enamel or with cement-mortar not less than $\frac{1}{4}$ in. in thickness. The use of steel pipe has certain advantages over cast iron. It has a lower initial cost, and calked joints, which usually require recalking with seasonal changes in temperature, can be avoided. Because the steel is lighter in weight, it can be more easily supported, and it is more adaptable to necessary changes.

Pipe connections to conduits and filters. All pipe connections to conduits and filters should provide for a wall flange located in the center of the wall, 6–8 in. larger in diameter than the outside diameter of the pipe, to act as a waterstop.

Where wall fittings are of cast iron, it is common practice to design the fittings so that the overall lengths correspond to the thicknesses of the walls in which the fittings are used and with bell ends on the sides of the walls to which piping is to be connected.

Where the wall fittings are of steel, a very satisfactory design has been used in which the wall fitting projects about 6 in. through the wall into the pipe gallery, with a heavy steel band welded to the pipe and machined to provide a shouldered end for a flexible leakproof coupling. This coupling, if installed with a $\frac{1}{8}$-in. gap between the pipe ends, permits some movement because of temperature changes and, at the same time, provides a joint which requires no maintenance to keep it tight.

Wash water pipe manifolds should be designed to resist hydrostatic thrusts. Where sleeve-type couplings or bell-and-spigot joints are used, shackles or harness sets should be provided unless other means are provided to prevent movement when the piping is under pressure.

Valves. It has been common practice in filter plants to install standard double-disc water works valves designed for 150-psi working pressure and 300-psi test pressure (*11*), although the actual working pressures encountered in a filter plant are quite low. Some newer plants are using large valves designed for a 50-psi working pressure with satisfactory results. When hydraulically operated, valves should be equipped with cast-iron cylinders lined with seamless red-brass tubing and should be provided with tail rods. Wherever possible, valves should be placed with the stems in the vertical position. Where it is necessary to mount a valve with the stem in a horizontal position, wide substantial bronze slides should be attached to the discs to carry the weights

of the discs, and heavy bronze tracks should be attached to the body and bonnet on which the slides may travel. Although rollers, rather than slides, are frequently attached to the discs, they soon wear flat in one spot and are of little use thereafter.

Riddick (*12*) reports the successful use of butterfly valves both for shutoff and as rate controllers. Weirs and float-controlled butterfly valves have been used as rate-of-flow controllers in industrial plants for many years.

The use of double-disc gate valves to control the flow of wash water in filtration plants is being increasingly recognized as undesirable. The double-disc gate valve is intended to be operated either in a fully open position or a fully closed position. Its use as a throttling device involves a chatter of the suspended gates, causing them to wear rapidly. It necessitates the complete reconditioning of the working parts in 5–10 years. The modern AWWA specification butterfly valve is fully acceptable for wash water control.

Sluice gates. In large plants especially, many sluice gates are generally used. The most common application of sluice gates in connection with filter units is in the waste wash water outlets. All sliding surfaces should be of good quality bronze. Sluice gate frames and gates should be constructed of heavily ribbed cast iron with heavy bronze seats dovetailed into the cast-iron frames and gates and pinned at close intervals. Adjustable side and bottom wedges and heavy bronze stems should be provided. The rectangular flat-type frame, which bolts directly to the face of a concrete wall, is most generally used, although the spigot-type frame is also used. If hydraulically operated, the cylinders should be of cast iron lined with seamless red-brass tubing and should be provided with tail rods. The setting should permit removal of the gate for repairs, and it is desirable that the frames be removable in case new seats are required. The frame and gate should be fitted and tested at the factory and should be installed as a unit rather than individually. Since it is difficult to sandblast the assembled unit without leaving sand on the sliding seats, the castings should be sandblasted and primed at the factory before assembly.

Pressure for hydraulically operated valves. Water under pressure for operation of hydraulically operated valves and sluice gates is usually supplied at pressures of 80–90 psi, but, if suitable pressures are not available, a hydropneumatic system or some other method must be provided. Sometimes only pressures below 80–90 psi are available in the distribution system. A small centrifugal booster pump (together with a standby unit) should then be installed for boosting the pressure. If a ¾-in. bleed line from the pump discharge is provided, the pump can operate against the closed hydraulic system without injury to the pump. Start-and-stop push buttons controlling the pump should be installed near the filter controls for each unit and running lights should be provided in a prominent location. Such an arrangement has operated successfully for several years at the municipal filter plant in El Centro, Calif.

In using a hydropneumatic system, certain advantages are to be gained over a closed system. The water drained from the cylinders is returned to a tank from which the pumps supplying the water to the hydropneumatic system take suction. The tank should be open to atmospheric pressure but should be kept covered to keep out

dust and dirt. A makeup supply line equipped with a float valve should be provided, and a low-cost soluble oil may be added to the water to lubricate the cylinders and various controls coming in contact with the pressure water. When this is done, scale formations and corrosion can be eliminated and much less maintenance is required. The closed hydropneumatic system is in use in Southern California at both the Metropolitan Water District's 200-mgd Frank E. Weymouth Softening and Filtration Plant at La Verne and at San Diego's 66-mgd Alvarado Filtration Plant.

It is desirable to use hard-drawn copper tubing for all piping related to water under pressure for operation of hydraulic cylinders because of its freedom from corrosion.

Any products of corrosion interfere with the proper operation of the controls and tend to score the cylinder walls. For this reason it is important to provide a strainer at the point at which the water enters the pressure system.

Large valves and sluice gates sometimes stick and are difficult to open with the available water pressure. This trouble can be avoided to a large extent if a pressure relief valve is provided in the pressure connection at the top of the cylinder to limit the closing pressure to about 35 psi, thus preventing the severe wedging action which may occur when the normal pressure is applied.

Unions should be provided at connections to all hydraulic cylinders.

9. Filter Controls

Operation of a rapid sand filter is accompanied by an increase in loss of head through the sand. This is caused by the collection of mud and floc on the sand surface which impedes the passage of the water being filtered. As it is impracticable to increase the operating head over the sand to offset the increasing loss of head, a uniform rate of flow through the filter bed can be maintained only by regulating the size of the opening in the effluent line and making it proportional to the head available for producing flow through it. Automatic regulation, such as is provided by rate controllers, is therefore essential to good filter plant operation.

The operating power used to manipulate the valves to perform this function may be hydraulic, pneumatic, or electric, and the controllers fall into two classes: direct acting and indirect acting (pilot operated).

Direct acting controllers. One type of controller is a direct-acting venturi-actuated controller which is designed for either horizontal or vertical mounting. The particular features of this type are a low head loss, lack of falloff after long runs, less weight, and less space for installation. It is the purpose of this controller to dissipate hydraulic head so as to maintain a constant rate of flow. This controller is designed using statically and dynamically balanced valve blades, thereby decreasing the possibility of binding action. The diaphragm pot is removable and may be serviced in place. The controller works on the theory involving the balance of moments as created on the controller beam. The balance of forces is created by setting the counterweights at a predetermined distance from the fulcrum balancing the venturi differential (as created by the flow through the venturi tube). The venturi differential is applied to the diaphragm pot of the controller, and the position of balancing is a func-

tion of the valve opening as determined by the piston load on the diaphragm pot and the position of the counterweight.

Another type of controller is a forerunner of the above type, but is designed to dissipate greater heads and is particularly useful on pressure filters. Because of design features, it can be used for tight closing, thus achieving a better control over flow.

Certain component parts are added to change the balance of forces directly, or, in some way, to induce a change in balance so as to perform the particular function for which the component part has been added. This change can be brought about by various combinations of the auxiliaries as follows:

a. Float shutoff
b. Fluid counterweight
[1] Direct connection to the portion of the system which is to be controlled
[2] Solenoid or mechanical valve control
c. Auxiliary diaphragm pot, either hydraulic or pneumatic
d. Electrically operated counterweights
e. Valves which depend on a constant pre-set elevation of a clear well.

Another type of controller is similar in principle and operation to the first controller mentioned, but it may be fitted with auxiliaries such as those listed above.

Indirect-acting controllers. For certain requirements, particularly where the line size is over 12 in. in diameter, the pilot-operated, hydraulically powered controllers have some important advantages. The operation is simple and positive. Plant water pressure is supplied to the opening or closing side of the hydraulic cylinder through a pilot valve, which accurately regulates the position of the control vane in response to the set rate. The construction of the pilot valve is such that the movement of the stem is proportional to the departure from the "set" rate—that is, if the controlled flow deviates from the set rate by a small amount, the pilot valve is just "cracked" to admit a small flow of water to position the control vane properly. If the deviation from the set rate is great, a larger quantity of water is admitted to the cylinder to effect rapid movement of the stem. As the vane nears the control point, the flow of water to and from the cylinder is gradually throttled to stop the movement without overtravel. These controllers operate as amplifiers to convert the small energy obtained by the venturi differential into a powerful mechanical force.

One controller of this type has a hydraulically operated gate valve control section and is used where tight shutoff is necessary. Another has a hydraulically operated cylinder positioning a butterfly valve.

Air-operated controllers. In the air-operated type, the controlled–air pressure system provides an instantaneous pneumatic activation for operating loss-of-head and rate-of-flow gages, level gages, rate-of-flow controllers, and regulation of liquid level. The basic elements of such a system are: [1] a regulated air supply at a specific pressure; [2] one or more converters which convert differentials (either linear as loss of head, or parabolic as rate of flow) to a linear controlled air pressure; [3] an instrument which is operated directly by the controlled air pressure or a mechanism operated by the controlled air pressure for actuation of an air-operated valve of a rate-of-flow controller.

The loss-of-head converter receives the differential across a filter on either side of the loss-of-head differential diaphragm which is mechanically linked through the converter lever system to the air control valve. The operation of this valve is the heart of the system. In the raised, or open, position air is admitted to the controlled–air pressure chamber; in the lowered, or closed, position it is shut off.

The filter level water pressure on the high side of the diaphragm chamber is isolated from the air system by a pivoting bellows and seal ring, mounted in a yoke. The yoke is the fulcrum of the converter lever system. The lever system is also connected to the center plates of the controlled–air pressure diaphragm at the bottom of the controlled-air chamber. At zero position, the weight of the diaphragm, its center plates, and the lever system are balanced by a spring tension adjusted by a manually operated knob. When the system is in equilibrium, the air control valve is closed. The zeroing adjustment insures that the control valve is accurately seated. The control valve stem has a movement of approximately 0.0015 in. The controlled-air chamber is provided with an orifice escapement always open to atmosphere. The orifice maintains the controlled air pressure within its operating range. The converter delivers a controlled air pressure of 0 psi at no loss of head, 3 psi at maximum loss of head, and intermediate pressures in direct proportion to intermediate differentials—for example, 1.5 psi at 50 per cent of maximum differential or loss of head. It functions as follows:

When there is no differential, the air control valve is closed and there is no controlled air pressure in the chamber, as the orifice escapement vents to atmosphere. The instant a differential is received by the loss-of-head diaphragm, the air control valve opens and builds up a pressure in the controlled–air pressure chamber against the orifice escapement until the pressure on the controlled–air pressure diaphragm balances the differential and the air control valve is correctly positioned. It remains in balance until greater differential causes the air control valve to open further and start a repetition of the cycle. The controlled-air pressure is therefore a direct measure of the differential or loss of head.

The rate-of-flow converter receives the differential measured by a venturi tube (or orifice plate) on either side of the rate-of-flow differential diaphragm. Through the converter lever system, the diaphragm is mechanically linked to the air control valve. It is the operation of this valve which makes the system work. In the raised, or open, position, air is admitted to the controlled–air pressure chamber; in the lowered, or closed, position, it is shut off. The operation of the controlled–air pressure chamber is similar to the loss-of-head converter except that the lever system is connected to a counterweight and paraboloid float which is totally submerged in mercury at zero level. The mercury well is connected to a riser tube vented to atmosphere and provided with a filler cap and drain plug. During operation the controlled air pressure displaces mercury in the mercury well. At zero level the entire lever system counterweight and float are balanced by the adjustable lever balance weight and the buoyancy of the float in mercury. When the system is in equilibrium, the air control valve is closed. The zeroing adjust-

ment insures that the control valve is accurately seated. The control valve stem has a movement of approximately 0.0015 in.

The rate-of-flow converter operates at the same pressures as the loss-of-head converter. It functions as follows: When there is no differential—that is, no flow through the venturi tube—the air control valve is closed and there is no controlled air pressure in the chamber, the orifice escapement venting it to atmosphere. The instant a differential is received by the rate-of-flow diaphragm, the air control valve opens and builds up a pressure against the orifice escapement. The air pressure acting within the mercury well depresses the mercury until the effective weight of the uncovered portion of the paraboloid float rebalances the lever system, and the air control valve is then correctly positioned. The system remains in balance until a greater differential on the rate-of-flow diaphragm causes the control valve to open further and repeat the cycle. The shape of the float automatically converts the parabolic flow differential function to a corresponding linear function. The controlled air pressure is therefore a direct measure of the rate of flow through the venturi tube.

The rate-of-flow control system consists of a pneumatically operated butterfly valve which is positioned by a rate-of-flow controller located in the operating table. The measurement of flow is achieved either by an orifice plate or venturi tube. The flow measurement to the signal controller originates in the square-root converter-transmitter or differential-pressure (d/p) cell transmitter located in the pipe gallery. Since the rate-of-flow controller is located on the operating table, the operator can read the rate of flow, change the filtration rate by setting the index of the controller, or manually position the valve to a fixed position.

The loss-of-head indicator is measured by means of the d/p cell transmitter which provides pneumatic signals proportional to the loss of head. A gage provided on the operating

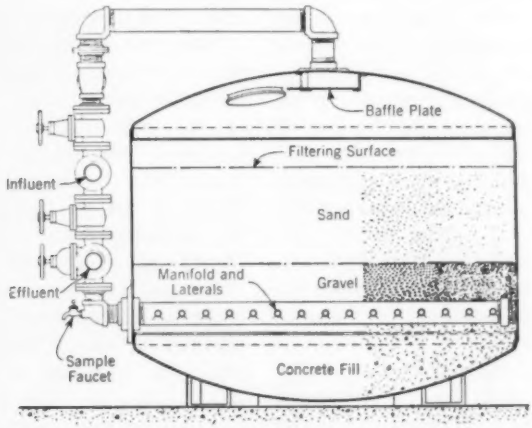

Fig. 11-8. Cross Section of Typical Pressure Filter

table receives this signal and indicates the loss of head in feet of water.

10. Pressure Sand Filters

Pressure filters are based on the same principle as gravity-type rapid sand filters, but the underdrains, gravel, and sand are placed in a cylindrical tank and the water is passed through the filter under pressure. In pressure filters the tank axis may be either vertical or horizontal. Filters of this type are intended for use where raw water is supplied under pressure and where it is desired to filter and deliver the water without further pumping. A cross section of a pressure filter is shown in Fig. 11–8.

Finished water from a pressure filter requires as careful pretreatment and as good a quality of applied water as would be necessary for finished water from free-surface filters. The extent and character of pretreatment are, of course, governed by the quality of the effluent desired.

Pressure filters are subject to the same operating difficulties as afflict standard rapid filters, except that air binding rarely occurs. Most of the troubles encountered are a result of careless operation or improper application of the water by the operator, both inherent in the nature of these filters. Their use for the production of a domestic water supply is not approved by some authorities. The important objections to pressure filters are listed by Charles R. Cox (*13*) as follows:

a. The treatment of water under pressure seriously complicates effective mixing, coagulation, and sedimentation of the water to be filtered and, thus, many pressure filters are installed without adequate coagulation facilities.

b. It is more difficult to apply chemicals to water under pressure.

c. The appearance of the water being filtered and of the sand bed is not under observation, nor is it possible to observe the effectiveness of wash water or the degree of agitation during the washing process.

d. Because of the inherent shape of pressure filters, it is difficult to provide wash water gutters effectively designed so that material washed from the sand is discharged to waste and not flushed back to other portions of the sand bed.

e. It is difficult to inspect, clean, and replace the sand, gravel, and underdrains of pressure filters.

f. The operation of such units under pressure encourages the pumping or forcing of water through the beds at excessive rates.

11. Diatomite Filters

The diatomite filter is a special type of pressure filter employing a processed diatomaceous earth as the filtering medium. It was developed primarily for military use during World War II, and has since been used to a limited extent for swimming pool waters and small potable supplies.

The filter is enclosed in a pressure tank with influent, effluent, wash water, and drain piping connections. The structural support for the filter medium is a fine metal screen, a synthetic fabric, or porous ceramic material. This filter is prepared for use by precoating, by which diatomaceous earth carried in suspension in water is built up in the filter base to a thickness of about $\frac{1}{8}$ in. The filter is then placed in service. The influent water is treated with a small amount of diatomite slurry to build up the filter medium as the filtration proceeds. Filtration rates range from 1 to 5 gpm or

more, depending on the character of influent water and results desired. After operation periods of several hours or days the filter is taken out of service, backwashed, and flushed. After precoating it is again returned to service.

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Reprints of 'Water Quality and Treatment' Revision

As a new edition of *Water Quality and Treatment* will not be published for at least several years, reprints of this revised chapter will be distributed without charge to future purchasers of the current (second) edition of the book, which may be obtained from the Association for $5.00 per copy ($4.00 to members paying in advance). Reprints will also be made available for purchase at a later date. Further details will be announced in a future issue of the JOURNAL.

Design and Operation Data on Large Rapid Sand Filtration Plants in the United States and Canada

Kenneth W. Cosens

A report prepared for AWWA Committee 8913 P—Revision of Water Quality and Treatment, *Chapter on Filtration, by Kenneth W. Cosens, Assoc. Prof., College of Eng., Ohio State Univ., Columbus, Ohio.*

AS a member of an AWWA task group on high-rate filtration which was later expanded and assigned the task of revising Chapter 11 of the AWWA manual *Water Quality and Treatment*, the author was given the project of bringing up to date the information on rapid sand filtration plants in the United States and Canada. The new information was to follow the general pattern used in the article by Eugene A. Hardin which was previously published in the JOURNAL (*1*).

To carry out this assignment, questionnaires were sent to plant personnel in all cities with populations over 100,000, as indicated by the 1950 census. Cities with metropolitan areas in excess of 100,000 were also included. The cooperation of the many plant superintendents who answered the questionnaires is greatly appreciated.

Reports were received from 79 plants, as compared to the 56 plants reported approximately 10 years ago (*1*), and the present classification is somewhat different, including a few more of the smaller plants. Where replies were not received, information was freely borrowed from the earlier survey.

No search of the literature was made to locate more complete descriptions of particular plants. The bibliography at the end of the earlier article (*1*) can be referred to for information on the plants listed there.

Explanatory Notes

Most of the figures in these tables need no explanation but, to make them more clear, a brief explanation of some of the items is given, together with notes on characteristics which do not fall within the limits of the tabulation.

Capacity figures are given in US gallons throughout.

The raw-water characteristics given are those of the water entering the treatment plant.

Chemical feed is designated as "dry" when the chemical dosage is measured in a dry condition and as "solution" when the dosage is of a given concentration of chemical solution measured in a liquid form.

Velocity of mixing for a baffled mix is the rate of flow through the length of travel given; for the hydraulic jump at Cleveland it is the velocity in the jump; for mechanical agitators it is the peripheral speed of the mixing paddle or blade; for tangential mix, no velocity is given.

Coagulation basin lengths and widths given are at the water surface, whereas the depths are either the range of water depth or the deepest water. Some peculiarities in shape are mentioned below.

The velocity in filter influent main or header is the maximum average cross-sectional velocity at nominal capacity flow.

The velocity in filter inlets is the velocity in the smallest cross-sectional area in the connection of filter to the influent main (or header) when the filter is operating at nominal capacity.

The dimensions of those filters which are divided into two sand areas by a center-wash gullet are indicated thus: 2 at $a \times b$, where a is the width and b, the length of sand area in each half.

Nominal capacity of plant is the product of the number of filter units and the size of units in million gallons per day.

Table 1 gives the characteristics of rapid sand filtration plants in the United States and Canada. Tables 2 and 3 list cities of over 100,000 population which have slow sand filtration plants, or do not have rapid sand plants, respectively.

Water Plant Notes

1. Akron, Ohio. The supply is impounded in a reservoir of 4.2-bil gal capacity on the Cuyahoga River. The capacity of the original filter installation in 1915 totaled 20 mgd. These filters are 29 ft long and have underdrain laterals cast in the floor under the Wheeler bottom construction. Additional filters of 10-mgd capacity were installed in 1919 (length 25 ft), and of 2-mgd capacity in 1922 (length 29 ft). The last two installations have false-slab construction under the Wheeler bottom. A mixing chamber and second coagulation basin were added in 1925. Previous to 1952, Palmer surface wash was installed in all 25 filter units and a new wash water tank with 125,000-gal capacity was installed. Distribution system storage was increased to 33 mil gal.

2. Albany, N.Y. The supply is impounded on Hannacrois Creek, where storage reservoir capacity is 12.1 bil gal, and Basic Creek, where storage reservoir capacity is 0.716 bil gal. This supply replaced the Hudson River supply in 1932. Filtered-water storage on the distribution system totals about 200 mil gal in three equalizing reservoirs. Albany also has slow sand filters of about 20-mgd total capacity, part of the old supply held in reserve for emergency. Baffling is accomplished by 4 in. $\times$ 2 in. openings in the baffle wall. Settled-water conduit is a concrete flume 11 ft deep $\times$ 4 ft wide. Filter inlets are 2 ft $\times$ 2 ft sluice gates. Floor strainers in the filters discharge in dual rows into 4 in. $\times$ 6 in. concrete laterals. Flume arrangement between settling tanks and filters make it difficult to apply lime or carbon to the settled water.

3. Allentown, Pa. This city has several sources of supply: 12 mgd from two springs with no treatment except chlorination; 30 mgd from Little Lehigh River which is filtered; and 3 mgd from two deep wells as a standby supply. The filters are surface washed with agitators. Sand expansion gages are used in both the old and new parts of the 30-mgd plant.

4. Atlanta, Ga. Preliminary sedimentation in the raw-water storage reservoir totals from 7 to 10 days. Filter data given are for gravity filters: seven 3-mgd units installed in 1923,

and one additional 3-mgd unit and six 5-mgd units installed in 1940–41. There are also 36 horizontal-type pressure filters of 0.5-mgd capacity each and 12 Hyatt pressure filters of 0.25-mgd capacity each, the latter being the original filters installed in 1887. The basin dimensions given are for the coagulation and settling basin built in about 1920. It has sloping sides and a hopper-shaped bottom. About one-third of the basin, at the entrance end, is used as a mud basin and may be cleaned independently.

5. Baltimore, Md. The supply is impounded on Gunpowder River. The present raw-water storage is approximately 23 bil gal. The coagulation and settling basins are somewhat irregular in shape and have sloping exterior sides. The line is added to the filtered water to reduce corrosive action.

6. Binghamton, N.Y. There are four 2.5-mil gal covered reservoirs and one 0.75-mil gal reservoir on the distribution system.

7. Birmingham, Ala. Wash water gullet locations at the Shades Mountain Filter Plant were at side and front. Those at the side were circular; the others were not. Lime is used only part of the time.

8. Buffalo, N.Y. Because there is no mixing chamber, the raw-water conduit is dosed with alum at the low-lift pumps and mixed in the 78 ft of conduit to the coagulation basins. New alum feeders and filter surface wash were being installed in 1941–42.

9. Cambridge, Mass. Preliminary sedimentation is obtained in impounding and storage reservoirs. Mixing is accomplished in the raw-water conduit to coagulation and settling basins. Entrance to coagulation and settling basins is through a wall perforated with 2-in. diameter holes. Filters are washed to a constant sand expansion which requires variations in rates of wash between the limits given. The filtered water is aerated. A 48-mil gal filtered-water reservoir on the distribution system is located at Payson Park.

10. Charleston, S.C. In addition to the use of low-service pumps for mixing, the water passes through 2,095 ft of 30-in. water main on the way to sedimentation basins. The sedimentation basins, built in 1904, are constructed of earth embankment with inside slope paved. The oval sedimentation basin contains 1.76 acres at normal flow level.

11. Charleston, W.Va. The original plant, built in 1906, was the one for which most data were given. Newer plants, built in 1918 and 1948, were of 7.6- and 5.6-mgd capacity, respectively. The 1906 plant had a capacity of 6.6 mgd, making the total for the three plants equal to 19.8 mgd. The 1918 plant has six filters, four of which have 1.1-mgd and two have 1.6-mgd capacity. The 1948 plant had four filters at 1.4 mgd each, with dimensions of 21 ft × 23.5 ft and depth of 8.7 ft. Sand surface area is 15.5 ft × 23.5 ft and filter box is 9 ft deep. Sand depth is the same for all plants. The sand in the 1918 filters has an effective size of 0.45 mm, except for two filters which are equipped with anthracite material with an effective size of 0.70 mm. There is 18 in. of gravel in the 1918 and 1948 plants, maximum size being 2.5 in. in the 1918 and 1.5 in. in the 1948 plant. Minimum gravel size is ¼ in. in all plants. The 1918 filters have perforated pipe laterals with manifold, while the 1948 filters have a Leopold bottom. Rate of wash water rise is 24 ipm in both 1918 and

1948 plants. Air is used in the earlier plant but not in the latter for sand agitation. Height of trough crest above sand is 24 in. in the latter two plants, and the clear distance between troughs is 5.5 ft and 6.08 ft in the 1918 and 1948 plants, respectively. Wash gullets are in the front position for the latter two plants.

12. Charlotte, N.C. Much of the operational data given were averaged for the years 1951–52. Data given relative to the mixing chamber applied to primary and secondary rapid mix only; the flocculators were not considered. Tangential mechanical mixing and the use of air were reported for type of mixing. The 9.25-ft depth of filter box included the false bottom. The strainer systems in the filters are Wheeler bottoms and modified Leopold bottoms. Palmer filter sweeps are used for surface washing.

13. Chattanooga, Tenn. The supply is treated in two parallel-operated purification plants, both of which employ split lime treatment applied to the settled water and to the effluent from the filters. Plant No. 1 comprises conventional coagulation, sedimentation, and filtration, with the first two sedimentation basins operating in parallel and delivery into the third basin. Plant No. 2 utilizes four 2.5-mgd purification units of the multiple-treatment type. Each unit consists of an open steel tank, 90 ft in diameter and 17 ft high, subdivided into three compartments by two concentric steel rings. The innermost compartment provides for mixing and coagulation and has a diameter of 30 ft, providing a detention period of 45 min at rated capacity. Raw water, to which the coagulant has previously been applied, is introduced tangentially at the surface of this compartment and is withdrawn near the bottom through eight baffled port-openings to the settling compartment. The settling compartment has an inner and outer diameter of 30 ft and 81 ft and provides for a detention period of approximately 5 hr. The flow through this compartment is radially upward with the settled water discharging into continuous collecting launders on both the inner and outer walls through series of equally-spaced 1½-in. orifices. A connecting trough is provided between the launders and a 12-in. × 5-ft vertical chute discharges the settled water into the filter compartment. Lime application is made at the entrance to this chute. The filter compartment comprises the outer 54-in radius of the purification unit and includes a 15-in. wide wash water gullet. The filter bed is 39 in. in width, having a surface area of 885 sq ft and is composed of 30 in. of sand directly supported by carborundum plates. Filtered water discharges into an annular compartment, 39 in. wide × 48 in. high, located beneath the filter section and from which flow is by gravity to the clear-water basin. Each unit is equipped with complete flow control actuated from operating panels installed in a control building which is centrally located with respect to the four units.

14. Chicago, Ill. The South District Filtration Plant serves about one-third of the city of Chicago. Ten of the 80 filters are operated at high rates as follows: four filters operated at 4.0 gpm per square foot, four at 4.5 gpm per square foot; and two at 5.0 gpm per square foot. Mechanical-type flocculating equipment used. The settling basins are of two-story depth. Settled-water filter inlet conduits are of 30-in. pipe, and 11.5 ft of water is maintained

in the 13-ft deep filter boxes. Wash water troughs are 32 in. wide.

15. Cincinnati, Ohio. This plant has preliminary sedimentation basins of 340-mil gal capacity with a surface area of about 43 acres and a maximum depth of water of 54 ft. Modern chemical feed and mixing and flocculation facilities were installed in 1936–38. The original filters were also completely remodeled and twelve new filters added at that time. Analysis data is for 1952. Under chemical feed, the 0.86 grains per gallon (gpg) reported in the space for alum actually is quantity of iron (Fe^{+++}) being added. There are two interconnected filtered-water reservoirs with combined capacity of 28 mil gal.

16. Cleveland, Ohio. At the Division Avenue Plant, the alum is measured dry by automatic scales which dump batches of a given size at predetermined intervals into a dissolver from which the alum is forced by an ejector to the raw-water dosing point. Mixing is accomplished by the use of a baffled chamber.

At the Baldwin Plant the prepared alum solution is mixed with the raw water by a hydraulic jump followed by a baffled conduit. The time at the jump is very short and the time in the rest of the mixing flume is about 1 min. The 600–700-fpm velocity given is the velocity in the hydraulic jump. The velocity in the baffled conduit is about 300 fpm.

Data relative to the large new Nottingham Plant recently placed in operation were not furnished.

17. Clifton, N.J. This plant is the headquarters for the Passaic Valley Water Commission, which serves several communities other than Clifton. Lime is added after filtration. Five-pass flocculators are used. Since 1942, plant improvements have been installed, consisting of flash mixer, clarifying and flocculating equipment, new Leopold filter bottoms, anthracite filter medium, pH control equipment, and breakpoint prechlorination. (See Item No. 57 for plant description.)

18. Columbia, S.C. The wash water troughs are located 6 ft center to center in the old plant. There are two filtered-water reservoirs, each at 3-mil gal capacity.

19. Columbus, Ga. In addition to over-and-under baffles, it was reported that there were "end-over-end" baffles in the sedimentation basins. A 20-in. main was used for filter influent while a 16-in. pipe served as a filter inlet.

20. Columbus, Ohio. The raw water is impounded in two storage reservoirs, totaling about 1.5 bil gal of available capacity, on the Scioto River. This plant is a lime–soda ash softening plant, 3.7 gpg of Na_2CO_3 being added for softening in addition to lime. Average raw-water hardness is 275 ppm and average output hardness is 100 ppm. The lime-mixing tanks are also stirred by mechanical agitators. The sedimentation basin effluent is recarbonated by a pipe grid placed in the bottom of the outlet end of the final basins. Besides the filtered water storage at the plant, two 1-mil gal standpipes and one 2-mil gal standpipe are located on the distribution system.

Not reported in the 1952 survey by the plant officials were three circular clarifiers, an additional lime–soda ash softening plant using only well water, new elevated storage, addition of sodium hexametaphosphate, and occasional chlorine dioxide treatment.

A new dam, under contract on the Big Walnut Creek, will impound 19.7 bil gal of water.

21. Corpus Christi, Tex. Settling time is 4 hr in two tanks and 10 hr in two other tanks in parallel. Wash water gullets are in the front in twelve filters and in the side of twelve other filters. Filtered-water reservoirs are constructed both as ground tanks and as elevated tanks, both being covered. Plant capacity is 33 mgd but is overloaded to 50 mgd.

22. Dallas, Tex. The supply is impounded in a 63-bil gal storage reservoir behind Garza Dam and in three other small impounding reservoirs of 0.8 bil gal combined storage capacity, all on Trinity River. Filters are washed by direct pump pressure with no wash water tank. Eight new filter units were added in 1936–40. A secondary mixing basin with a 4-min retention period is located between the settling basins and the filters. About 80 per cent of the water used in Dallas is treated at this plant; the remaining 20 per cent, taken from deep wells, is not treated. The 1.0 gpg of alum reported is actually $FeSO_4$.

23. Denver, Col. In the Marston Lake Plant some presedimentation is obtained in impounding Marston Lake on the South Platte River. The raw water is aerated by spray nozzles, the water falling into a mixing chamber. Anthracite coal is used as the fine filtering material in place of sand. Filters are washed at a 3.5-ft loss of head in this low-head plant. The 0.05–0.50 gpg of lime reported is actually $Na_2Al_2O_4$.

The Moffat Plant at Denver treats water diverted from the Fraser River or from the headwaters of the Colorado River through the Continental Divide via the Moffat Tunnel.

24. Des Moines, Iowa. A softening plant was placed in operation as an addition to existing pumping-station facilities which had been in operation since 1920. Analysis data given are for the year 1951. The settling basins are two-level, the upper being 15.3 ft deep and the lower being 12.4 ft deep. In addition to the sand and gravel, there is a 5-in. layer of torpedo sand varying in size from 0.8 to 1.2 mm. A fixed-type surface wash is provided. Plant pumping capacity is 100 mgd. Sludge return to mixing chamber has been of great benefit in reduction of chemicals used and in increasing the length of filter runs.

25. Detroit, Mich. At the Water Works Park Plant, chemicals are dosed ahead of raw-water pumps and mixed in these pumps. This is followed by a 3-min baffled-chamber treatment at the entrance to the coagulation basins.

At the Springwells Plant, mixing is by baffle walls and two mechanical agitators. Filtered-water storage of about 10 mil gal is obtained in six elevated tanks on the distribution system. Distribution or guide vanes are used at both plants to aid in baffling of settling tanks.

26. Edmonton, Alta. This plant is a lime–soda ash softening plant. An obsolete plant not described may be used to supplement peak flow conditions.

27. El Paso, Tex. Two primary clarifiers are 80 ft × 80 ft × 14 ft; the aerator is 70 ft × 15 ft × 15 ft; 2 flocculating machines are 36.5 ft × 67 ft × 12 ft; two settling basins are 105.5 ft × 67 ft × 12 ft, each with 750 lin ft of weir per basin; two flocculating basins are 78.5 ft × 31.5 ft × 14 ft. There are two settling basins of which dimensions are not given. Total head loss from primary tanks to water on filters is 5.71 ft. There is 70-mil gal

storage on the distribution system at three levels, using five separate pressure systems. One-half minute of flash mixing with mechanical paddles occurs prior to flocculation. Underwater gas burners provide recarbonation. Aeration of raw water is done before chemical treatment. Filter influent channels are 24-in. × 54-in. flumes in one part of the plant and 30-in. steel pipe in another. Filter influent pipes are 12-in. cast iron.

28. Erie, Pa. In the Chestnut Street Plant the filters have 20 in. of sand and 7 in. of anthracite material, while, at the West Plant, eight of the filters are the same. Six other filters have 30 in. of anthracite material and no sand.

29. Evansville, Ind. Filtered-water storage on the distribution system is of 20-mil gal capacity. Plant additions in 1939 consisted of four filters and new chemical feeders.

30. Flint, Mich. The plant was adapted to softening in 1939 by the addition of one primary mixing tank, three coagulation basins, and one mechanically cleaned sedimentation basin, together with new chemical feed equipment. Additions since 1942 are not reported.

31. Fort Wayne, Ind. The raw-water supply is impounded in a storage reservoir of about 700-mil gal capacity on the St. Joseph River. The plant softens the water with lime and soda ash as well as clarifying it. Maximum raw-water hardness is 475 ppm, 300 ppm being the average. Average output hardness is 100 ppm. Settling basins are of the two-story type.

32. Fort Worth, Tex. The general pattern of water treatment at this plant is about 10 min of quick mix after the addition of lime and alum, followed by 35–50 min of gentle mixing in hydraulic or mechanical flocculators, and then a little over 4 hr of settling. Thirty-nine million gallons per day pass through 24 filters with cast-iron underdrain, while 40 mgd pass through twenty filters with Leopold bottoms. Chlorine is added to the filtered water before storage in a 10.742-mil gal clear well having four compartments.

Presedimentation takes place in the 15-bil gal impounding reservoir at Lake Worth, on the West Fork of the Trinity River. The inlet to the sedimentation basins is behind a curtain wall or down-baffle and the outlet is over a skimming weir.

33. Gary, Ind. Until 1952, Gary's water supply had not been filtered. Lake Michigan water was chlorinated and pumped to consumers. Data given are for the filtration plant under construction which is expected to go into operation in 1953. Note that data are given for two types of operation: 2-gpm per square foot and 3-gpm per square foot filtration rates.

34. Grand Rapids, Mich. The plant clarifies Lake Michigan water. A grit basin precedes the mixing chamber. The five filters added in 1930 are 20.5 ft × 42 ft in size and filter at a lower rate (about 100 mgd per acre). Storage on the distribution system totals about 15 mil gal.

35. Hamilton, Ont.

36. Hammond, Ind. During peak load the plant described is operated at filtration rates as high as 3.75 gpm per square foot. Maximum summer rate reported was 36 mgd per acre. Surface wash has stationary jets. Rate controllers were changed from original installation of 4-mgd to 5-mgd capacity.

37. Indianapolis, Ind. The supply for the White River Plant is impounded

at Broad Ripple on the White River, with very small storage behind the diversion dam. The coagulation basins have sloping sides with paved slopes. Indianapolis has six slow sand filters of 36-mgd capacity, of which three are now retired, on the White River supply, and some well water is used.

The Fall Creek Plant is supplied from an 8-bil gal impounding reservoir on Fall Creek.

Conduits and basins of the 36-mgd plant on White River were designed for a 72-mgd flow. Figures given are for velocities calculated on number of basins required for only 36 mgd.

38. Kansas City, Kan. This plant also has two primary basins for preliminary sedimentation before the raw water is dosed with chemicals. The coagulation basins are operated in series and are allowed to fill up with sediment to within 3 ft of water surface before cleaning, at which depth the time of retention is about 6 hr. Filtered-water storage on the distribution system amounts to 13 mil gal. Averaged raw-water turbidity is 2,040 ppm.

39. Kansas City, Mo. The raw water receives preliminary sedimentation in four 200-ft diameter circular tanks with mechanical clarifiers before being dosed with chemicals. Initial mixing is in hydraulic jump channels recently built across the two 90-ft diameter original tangential-mixing chambers. The plant was adapted to softening in 1941 by the installation of rapid-mix channels, and flocculating and sludge removal equipment in existing basins. A new chemical building and chemical handling and feeding equipment were also added. Reservoirs on the distribution system hold 62 mil gal in storage. Average turbidity is 2,000 ppm.

40. Knoxville, Tenn. Two circular mixing chambers with mechanical stirrers are operated with tangential flow at velocities up to 2 fps. Water enters tangentially at the tops and exits from the center of the bottoms. The basin flow is straight through from the west to the east end. Widths of basin passes vary and basins have sloping sides, the length and width dimensions given being those of the average water surface. Basin inlets are located behind a fence of wooden slats and the basin outlets are skimming weirs. The storage on distribution system is 10 mil gal for equalizing and 1 mil gal for reserve. Lime is applied only to settled water ahead of filters for pH adjustment.

41. Little Rock, Ark. Four 2-mgd sand filters were added in the 1948 construction program. Flash mixers precede flocculation. The plant filters water at an annual average rate of 15 mgd. Filter bottoms are of three types; four units have pipe laterals, four have Leopold bottoms, and fourteen units have false steel bottoms with brass strainers on 6-in. centers. Backwash rate is 20 ipm for the ½-mgd units where freeboard (top of sand to trough crest) is 24 in. The backwash rate, however, is 30 ipm for the 2-mgd units where freeboard is 36 in.

42. Los Angeles, Calif. This tabulated report is that of the Metropolitan Water District of Southern California. The 1940 population of 2,028,000 represented thirteen member cities and the 1950 population of 3,352,000 represented sixteen member areas containing 914 sq miles. The district does not supply all water used within the district. During 1951–52 they softened and filtered 90,000 acre-ft and delivered a total of 197,000 acre-ft. Ac-

tivated silica is used for coagulation. The dosage is 0.174 gpg as SiO_2 and 0.069 gpg as Cl_2. Alum is used only at times of high turbidity and is used only in conjunction with activated silica. Lime is used as a filter aid in conjunction with activated silica. The original design of settling tanks was center feed, radial flow to overflow weir around entire periphery. At nominal flow of 50 mgd, the velocity at 50 ft is 1.05 fpm and, at 100-ft radius, is 0.5 fpm. The new design calls for end feed with up-and-over baffle. At 50-mgd flow, the velocity on lower deck is 3.19 fpm and upper deck 2.25 fpm. Filter sand is laid in two layers. The lower 4 in. has an effective size of 1.0–1.5 mm and a uniformity coefficient of 2.0. The upper 20 in. has an effective size of 0.45–0.50 mm, and a uniformity coefficient of 1.65. Filter wash water is recovered. There is no reservoir at the filter plant, but three reservoirs on the distribution system have a combined capacity of 396 mil gal. Under construction are an additional 489 mil gal of storage on the distribution system.

43. Louisville, Ky. Preliminary sedimentation in two storage reservoirs of 55-mil gal capacity each, or about 1 day's retention time, precedes dosing of raw water. There is no regularly designed mixing chamber. Raw water is dosed in a slotted controller and is discharged through a sluice gate into each basin. The coagulation basins were revamped in 1929. The six original filters, built in 1909, were reconstructed in 1931. A 30-mil gal filtered-water storage reservoir is located about 12 miles from the filter plant, acting as an equalizing reservoir on the distribution system. Surface wash was installed in the filters in 1940.

44. Memphis, Tenn. Water is pumped from artesian wells by the Parkway Pumping Station and the new Thomas H. Allen Pumping Station. Prior to 1953 the only treatment was iron removal on filters. The filter box depth of 6.96 ft is to the top of gullet. Filter sand is composed of 3 in. of coarse sand 1–3 mm in diameter and 21 in. of filter sand with effective size of 0.50 mm.

45. Miami, Fla. Water hardness is 250–260 ppm. The Hialeah plant is outstanding because of the lime reclamation processes used. Water characteristics make it feasible to reclaim lime and to reuse the product.

46. Milwaukee, Wis. Settling is accomplished in two-story basins. The wash water tank is located on a bluff across the highway from the plant.

47. Minneapolis, Minn. At the Columbus Heights Plant one of the two old 47-mil gal raw-water storage reservoirs was used as a presedimentation basin and the other was covered and used as a filtered-water reservoir, flow from which is by gravity to the part of the city served. The 19.5-mil gal reservoir was completed in 1951, the 18-mil gal one in 1953 and the 15.2-mil gal one in 1952. The water-softening plant operates in connection with the filtration plants.

48. Mobile, Ala. Baffles are installed at outlet of mixing basins. Mechanical mixers are installed but are not used. Baffling is accomplished in the sedimentation basins by holes in a stilling wall and weirs at the discharge end. A wash water pump is used for backwashing filters. Filtered-water reservoirs consist of a 1-mil gal closed tank and two 10-mil gal open tanks.

49. Montreal, Que. The population served includes that of Montreal and

adjoining municipalities. The old slow sand filters were abandoned in 1935. Air, at a rate of 4 cfm per square foot, is used for agitating the sand in washing the filters. Three reservoirs float on the distribution system; one of 37-mil gal (Imp.) capacity; one of 60-mil gal (Imp.); and one of 2-mil gal (Imp.).

50. Nashville, Tenn. Turbidity of raw water averages 65 ppm and alkalinity averages 75 ppm. Perforated inlets are placed in the sedimentation tanks. The raw water is given ½ hr presedimentation in a grit chamber before dosing. Filtered-water storage on the distribution system totals 50 mil gal. An addition is being constructed which will provide two more filters of the same type, a Walker clarifier 75 × 188 ft, and Palmer surface wash in all filters.

51. New Orleans, La. Grit basins are provided for preliminary sedimentation. Filter underdrains are of seamless brass tubing, with the orifices pressed in to catch some impact of velocity. The plant has a capacity of 7.5 mgd and the wash water pump has a capacity of 6,600 gpm.

52. Norfolk, Va. Alum is applied as a solution while lime is applied dry. Each plant has the benefit of presedimentation in raw-water impounding reservoirs. The Lake Prince Reservoir has about a 4-bil gal storage capacity. New plant additions were under construction in 1942.

The coagulation basins at the Moore's Bridges Plant are of irregular width and have sloping sides. The dimensions that are given are of average water surface.

At the Lake Prince Plant the water is aerated by a waterfall immediately after mixing in a closed mixing chamber and is discharged from the aerator into the coagulation basins.

53. Oakland, Calif. The supply is impounded on the Mokelumne River. The plants supply the nine cities which comprise the East Bay Municipal Utility District—Richmond, El Cerrito, Albany, Berkeley, Emeryville, Oakland, Alameda, Piedmont, and San Leandro. The distribution system is interconnected between the San Pablo Plant, located at North Berkeley, and the San Leandro Plant, located at East Oakland. The filter underdrains at the upper San Leandro Plant are of brass tubing. These filters are washed one-half section at a time.

54. Oklahoma City, Okla. Turbidity varies between 20–400 ppm and alkalinity varies from 30–250 ppm. The supply is impounded in a 5.5-bil gal storage reservoir on the North Canadian River. The plant consists of primary aerator, primary mixing chamber, coagulation basins, recarbonation at basin outlet, secondary mixing chamber, filters, 1-mil gal clear well, secondary aerator, and a 2-mil gal clear well. The plant softens as well as clarifies the raw water. The secondary mixing chamber has a 20-min retention and uses a rapid mix by three motor-driven agitators. An elevated tank of 0.5-mil gal capacity floats on the distribution system.

55. Omaha, Neb. The plant has six large coagulation and settling basins of varying sizes, with a total capacity of 80 mil gal. Parts of these basins are used for dosing and mixing. The filters may draw water from any of these basins. The 5 ppm color reported is for finished water. The mixing chambers are: primary, 260 ft × 30 ft and average 9 ft deep; the secondary are 155 ft × 80 ft and average 8 ft deep.

Lime is added dry and alum is added in solution. Mixing time is 10 min in primary and 15 min in secondary tanks. After primary settling, turbidity is reduced to 324 ppm. Coagulated and settled water to the filters has a turbidity of 3.65 ppm. Prechlorination takes place midway in the presedimentation process just before addition of alum and lime in a mixing basin. After settling, a second application of alum is made. Ammonia was applied but has been discontinued since 1950.

56. Ottawa, Ont. Surface wash is by Palmer filter sweeps.

57. Paterson, Passaic, and Little Falls, N.J. District. This plant supplies Paterson, Bayonne, Passaic, Clifton, and several other New Jersey towns and municipalities. The plant has no regular mixing chamber, raw water being dosed as it enters the coagulation basins. The 1902 filters have wash gutters around the walls as well as across the center of the filter layer. The 1919 filters have central gutters with a front gullet. The Wanaque water supply includes 27 pressure filters of 0.75-mgd capacity, without coagulation or pretreatment. The distribution system includes three reservoirs, one each of 150-, 30-, and 3-mil gal capacity. (See Item No. 17 for additional plant description.)

58. Philadelphia, Pa. Each plant has slow sand filters in operation. A third slow sand plant of 24-mgd capacity is located on the Schuylkill River, and Torresdale slow sand filters of 200-mgd capacity are located on the Delaware River. The three storage reservoirs on the distribution system have capacities of 600, 70, and 40 mil gal.

The coagulation basins at the Belmont rapid sand plant are irregular in shape. Plan view dimensions given are of average water surface.

59. Portsmouth, Va. The water is obtained from two lakes and a well. Surface wash for the filters is of the Palmer type. The well source supplies 2.7 mgd of water into the clear-water basin. This well supplies sufficient high fluorine content to maintain 1 ppm of natural fluorine in the water delivered to consumers.

60. Providence, R.I. For coagulation, 0.67 gpg of 70 per cent ferric sulfate is added. Studies in high-rate filtration are being conducted. Some sedimentation is obtained in a 37-bil gal impounding reservoir. The raw water is aerated as it enters the plant. Ferric sulfate is applied and mixed in a raw-water conduit 825 ft long at a velocity of 15 fpm. Hydrated lime is added at a point 270 ft before entrance to the 35-ft diameter mixing tank, which has rotary flow with a tangential bottom inlet and a tangential top outlet. The irregularly shaped sedimentation basins were reshaped, deepened, and lined with concrete and four new filters were added in 1939–40. The filtered water is aerated by the available unused filtering head on spray nozzles. There is no clear-water storage at the filter plant, and the filtered-water storage is on the distribution system in the 39-mil gal Neutaconkanut low-service reservoir and the 12-mil gal Longview high-service reservoir, both of which are covered. This rapid sand plant replaced the original slow sand filters at Providence.

61. Reading, Pa. This plant supplanted two of Reading's four small slow sand filter plants. The settling basins have perforated brick walls across inlet and outlet ends. They may be operated in parallel or in series.

62. Richmond, Va. Both baffles and mechanical mixers are used in the mixing chambers. The raw water is polluted by sulfite wastes from paper pulp mills upstream. The plant has two presedimentation basins, of about 85-mil gal capacity each, which, used singly, give about 4 days settling. Additional mixing of 70 min is provided by mixing paddles installed in entrance end of sedimentation basins. Sedimentation basins are of somewhat irregular shape, but about the same volume. The dimensions given are averages. Filter inlets are at the rear of the filters. Filtered-water storage by gravity is 3 mil gal, but, by pumping, an available storage space of 7.5 mil gal may be used. Filtered water is aerated by pumping through spray nozzles during the 9 warm months of the year. Occasionally, chlorinated copperas (2.5 gpg) is used instead of alum. Prechlorination (1.2–7.5 ppm.), with dechlorination by sulfur dioxide to a residual of 0.2 ppm applied to filters, is practiced. Finished water is corrected by lime to a pH of 8.7 and disinfected with chlorine and ammonia.

63. Sacramento, Calif. To the original plant of 32-mgd capacity, eight filters were added in 1927 and additional preliminary treatment was added in 1932, the last addition being made in 1937. The new preliminary treatment works consist of a grit chamber 50 ft × 50 ft and 4 ft deep, a new aeration field of nozzles; four mixing tanks, 56 ft in diameter × 23 ft deep, with pump impeller–type mechanical mixers; two sedimentation basins, 100 ft square × 10 ft deep, with sludge collecting equipment; and 4-in. settling equipment, 44 ft × 145.5 ft and 9 ft deep, with intermediate sloping shelves and sludge-collecting equipment.

64. Saginaw, Mich. The enlargement in 1948 amounted to changing the source of supply from the Saginaw River to Lake Huron, greatly improving raw-water quality. The 6.5-mil gal settling basin includes 0.5-mil gal recarbonation chamber now used only for settling. The present effective size of sand is large because of carbonate coating.

65. Saint Paul, Minn. Baffles in the sedimentation tanks are around-end in secondary basins. Primary settling basins are open, secondary are covered. About 95 per cent of supply is obtained from the Mississippi River. This is pumped into a chain of fourteen small natural lakes partly supplied by springs which also add to the supply. Presedimentation is obtained in these lakes. Mixing is assisted by compressed air diffused into the water as it enters the mixing chamber. This accomplishes some aeration as well as thorough initial mixing of chemicals. The plant was adapted to softening in 1939 with the addition of a flocculating basin and a mechanically cleaned clarification basin. The sedimentation basins are normally used in series and the velocity figure is for series operation. These basins are now used for secondary settling. Six new filters were built in 1939, three of which were equipped and put into service. Two filtered-water reservoirs, of 16-mil gal and 18-mil gal capacity, float on the distribution system.

66. San Diego, Calif. Two of the three plants are pressure filter plants. Instead of alum, 0.63 gpg activated silica as sodium silicate and 0.42 gpg of ferric sulfate are added. In addition to the mixing reported in Table 1, 191 ft of mixing channel is equipped with mechanical agitation. Velocity is 61.3 fpm in this channel.

67. San Jose, Calif. The California Water Service Company operates the two filter plants described in Table 1 in addition to a treatment plant in Stockton which consists essentially of aeration, chlorination, and the feeding of sodium hexametaphosphate. There are no filters. The Mallard plant mixing chamber consists of two steel tanks in series. The backwash troughs in the Petaluma plant are joined in a V in circular filter.

68. Savannah, Ga.

69. St. Louis, Mo. In the Chain-of-Rocks Plant, raw water passes through a grit chamber for presedimentation before dosing. Lime is added at the entrance to the mixing basin and iron sulfate (0.73 gpg) at the exit of the mixing basin. The sulfate mixes in the conduit to the settling basins. Six settling basins are used in series. The water from these is dosed with alum at the venturi meters before entering a secondary coagulation chamber, where some mixing and coagulation is obtained in a pair of basins, operated in parallel. It is then applied to the filters and after chlorination, flows to Raden and Bissell's Point for storage in 20- and 54-mil gal reservoirs, respectively. Compton Hill reservoir provides 85-mil gal storage on the distribution system in the area served.

In the Howard Bend Plant, raw water is pumped to two 150-ft square presedimentation basins, equipped with clarifiers, for 2-hr sedimentation before dosing. Lime is added in a rapid-mix basin of the around-the-end type before entering the four tangential mixing chambers operating in parallel. Iron sulfate is dosed at the entrance to the tangential-mixing chambers. The mixed water enters two 150-ft square coagulation basins equipped with clarifiers. These basins operate in parallel and have a retention period of 2 hr. From these basins, the water flows to two final sedimentation basins operated in parallel for 8.7 hr of final settling. After final settling, it passes to a recarbonation basin where, after recarbonation, alum is dosed for secondary coagulation before the water is applied to the filters. A 5-mil gal clear well between the filters and the high-pressure pumps provides for flexibility in pumping to the 100-mil gal Stacy Park reservoir on the distribution system for the area supplied by this plant.

70. Tampa, Fla. The impounding dam washed out in 1933, and the water is now simply diverted. The original propeller-mixing equipment was removed in 1935.

71. Toledo, Ohio. The old supply from the Maumee River was replaced by a supply from Lake Erie, and a new filter plant was built in 1942. Surface wash is of the fixed type over the filters.

72. Toronto, Ont. The Island Filtration Plant has no regular mixing chambers or coagulation basins, but raw water is dosed at pump and is mixed in the pipes conveying it to filters. The filters are of the "drifting-sand" type. The two main wash water gutters for each filter are in concentric circles connected radially by six lateral gutters equally spaced. Sand is continually and slowly removed, washed, and returned. The outputs of the drifting-sand filters and slow-sand filters are mixed and pumped into the same mains. Raw-water turbidity averages 6 ppm and pH averages 7.8. The Victory Park plant was completed in 1941. Toronto has two storage reservoirs, one of 40- and one of 60-mil gal capacity, on the distribution system.

73. Trenton, N.J. Sudden changes in character of the raw water are frequent—that is, turbidity may change from 0–1,000 ppm in less than 1 hr. Flocculation equipment was added in one settling basin in 1940.

74. Tulsa, Okla. Some presedimentation is obtained in the 20-bil gal impounding reservoir at Spavinaw Lake, and some in the 500-mil gal Mohawk Lake plant. The raw water is aerated by injecting compressed air through porous plates in the bottom of the raw-water conduit. The filter underdrains are of seamless brass tubing with orifices pressed in to catch some impact of velocity. A 10-mil gal filtered-water storage reservoir floats on the distribution system. There are three mixing basins with two more under construction. Mixing is accomplished by a tangential slot, the water entering the mixing basins through openings 7 ft high and 12 in. wide. There are two additional settling basins under construction, and 20 mgd of settled water passes through a 54-in. pipe and 4 mgd through a 24-in. pipe.

75. Washingon, D.C. The data are reported by the Delecarlia filter plant. Some presedimentation is obtained in the 100-mil gal Delecarlia raw-water storage reservoir. Lime is added to raise the pH of the filtered water to prevent corrosion. The two mixing chambers may be operated in series or parallel. The longitudinal around-the-end baffles in the coagulation basins are slightly oblique. Wood-slot bottoms in the filters were replaced with perforated-pipe underdrains in 1941–42. The full filtered-water reservoir throttles the filters. Four other reservoirs are on the distribution system. A slow sand plant supplies the low-service areas of the city by gravity. The plant was expanded in 1942 and in 1949–52. Some of the new filters have anthracite material 22–25 in. in depth. Four filters have Leopold bottoms, eleven have porous-plate bottoms, and one has asbestos-cement lateral bottoms. Palmer surface sweeps were installed on 20 filters in 1943. A two-story floc-sedimentation basin was placed in operation in 1949. The six new anthracite material filters were placed in operation in 1952. A new 30-mil gal filtered-water reservoir is under construction.

76. Wichita, Kan. Turbidity of 20 ppm is caused by iron after aeration. For coagulation, 0.2 gpg of copperas is added. This plant softens a new well supply developed in 1940. The average raw-water hardness is 160 ppm; finished-water hardness is 120–130 ppm.

77. Wilmington, Del. The impounded supply serves through two filter plants, one slow sand and one rapid sand type. Part of the raw-water supply passes through a 35-mil gal presedimentation reservoir and a 12-mil gal slow sand plant, to a 6-mil gal filtered-water reservoir. The other part passes through the rapid sand plant. Four 2-mgd filter units were added to the rapid sand plant in 1939. The system includes a 40-mil gal filtered-water reservoir on the low-service distribution system and a 7.5-mil gal emergency reservoir on the high-service distribution system. A new filter plant is under construction.

78. Windsor, Ont. Two filter units on opposite sides of the operating gallery are operated from the same table. The filter gravel is bonded together with cement grout. The two mixing chambers are baffled to give decreasing velocities. The length of travel

and velocity of mixing are those of the mixing chambers proper. The time of mixing includes time in pumps and conduits preceding these chambers. An additional 2.5-mil gal filtered-water reservoir was added in 1941. Two new filters were added in 1952 as well as surface wash by Palmer agitators.

79. Youngstown, Ohio. This plant softens and clarifies the new Meander Creek supply. The old plant on Mahoning River supply was abandoned in 1932–35. In addition to the 1.1-mil gal clear well at the plant, the system includes standpipes of 3-mil gal capacity on the distribution system and a distributing reservoir of 30-mil gal capacity. Palmer surface wash mechanisms are used for washing filters. This plant is that of the Mahoning Valley Sanitary District and supplies water to both Niles and Youngstown, Ohio.

Reference

1. HARDIN, EUGENE A. Design and Operation Data on Large Rapid Sand Filtration Plants in the US and Canada. *Jour. AWWA*, **34**:1847 (Dec. 1942).

Key to Table 1

(a) Also preliminary sedimentation
(b) Also storage on distribution system
(c) Impounded supply
(d) Data given under any date is for the portion constructed in that year
(e) Also has 48 pressure filters as follows: 36 with capacity of 0.5 mgd and area of 175 sq ft, and twelve with capacity of 0.25 mgd and area of 87 sq ft (Hyatt)
(f) "Drifting-sand" filters with cemented gravel
(g) Also around-the-end baffling
(j) Also secondary coagulation after settling
(k) Mixing also in 825-ft conduit for 11.6 min at 71 fpm velocity
(m) No reservoir at filter plant; 39-mil gal reservoir on low-service system; 12-mil gal reservoir on high-service system
(n) Total population as of 1950 U.S. Census
(o) Also recarbonation
(p) Also aeration
(r) Also has a slow sand plant in operation
(s) Population connected as of 1952
(t) Split dose between settled and filtered water, with 0.5 grains per gallon (gpg) going to settled and 0.15 gpg going to filtered water
(u) 10.55 in. with air, grid at top of gravel
(v) Wash water troughs are common to two filters and main troughs lead to sewer

Types of Strainer Systems

A—Perforated brass plates over ridge and valley bottom
B—Manifold with perforated pipe laterals
C—Manifold with pipe laterals and brass strainers
D—Wagner bottom
E—Wooden slat grid
F—Wheeler bottom
G—Half-round cast-iron pipe raised ½ in. from floor leaving slot orifice between edge of pipe and floor
H—Perforated false bottom slab with strainers
I—Leopold bottom

TABLE 1. *Characteristics of Large Rapid Sand Filtration Plants in US and Canada, 1952**

	(1)	(2)	(3)	(4)
City	Akron, Ohio	Albany, N.Y.	Allentown, Pa.	Atlanta, Ga.
1. Population supplied (1950)	280,000	130,500	103,000	331,314 (n)
2. Population supplied (1940)	244,000		96,000	302,500
3. Date of beginning operation	1915	1931	1928 1953	1887
4. Date of enlargement	1912-22			1923 1938-40
5. Source of supply	Cuyahoga River	Hannacroix & Basic Creek	Little Lehigh River	Chattahoochee River
6. Average analysis of raw water:	(c)	(c)		(c)
6a. Turbidity—*ppm*	11-12	1.5	15	25-4400 (avg 234)
6b. Alkalinity—*ppm*	60-72	43	140	7-24 (avg 13.2)
6c. pH value (avg)	7-8	7.6	7.4	6.9
6d. Color—*ppm*	45-49	13	10	0
7. Mixing chamber:		(p)		
7a. Type of chemical feed	dry & solution	dry	dry	dry
7b. Alum added—*gpg*	1.90	1.5	0.8	0.59
7c. Lime added—*gpg*	0.37	0	0	0.26
7d. Mixing time at nominal capacity—*min*	2	20	0.3	23
7e. Velocity at nominal capacity—*fpm*	90	60		64
7f. Length of travel—*ft*	180	1,200		1,664
7g. Type of baffling	around end baffles	around end baffles	mechanical flash mix	around end baffles
7h. Type of mixing				
7i. Head loss in mixing—*ft*	1.5	3.0		0.8-1.3
8. Sedimentation basins:				
8a. Number of basins	6	3	2 2	4 @ 2, 2 @ 4, & 1 @ 5 mil gal (a)
8b. Settling time at nominal capacity—*hr*	2.0	2.25	2 4	9.3
8c. Velocity at nominal capacity—*fpm*	2.1	1.5		0.7
8d. Method of baffling	around end	perf. brick wall	none none	
8e. Turbidity of settled water—*ppm*	6	0.5	4-100	13
8f. Basins open or covered	covered	covered	open open	open
8g. Length of one basin—*ft*	125.25	196	110	1 @ 400, 2 @ 417
8h. Width of one basin—*ft*	54.33	50	113	1 @ 120, 2 @ 100
8i. Depth of one basin—*ft*	13	10-18	16 18	1 @ 24, 2 @ 14
8j. Total basin capacity—*mil gal*	4.5	3	3.356	21
9. Settled-water conduits:				
9a. Velocity (filter influent main)—*fps*	1.0	1.38		3.42
9b. Velocity (filter inlets)—*fps*	2.25	1.0		1.48
10. Filters:				
10a. Rate of filtration—*mgd/acre*	125	125	10 20	1923 (e) 1940 125
10b. Number of units	25	8	8 8	8 6
10c. Size of units—*mgd*	2	4	1.25 2.5	3 5
10d. Dimensions of sand bed—*ft*	2 @ 12 × 29 2 @ 14 × 25	2 @ 15.25 × 46.5	20.5 × 22 2@30×30	2 @ 12.37 × 42 2 @ 19.5 × 44
10e. Depth of filter box—*ft*	8.5	12.25	8 8	11
10f. Thickness of sand—*in*	30	30	20 30	24 27
10g. Effective size of sand—*mm*	0.55	0.33	0.40-0.45 0.48-0.55	0.40 0.50
10h. Uniformity coefficient of sand	1.29	1.75	1.57 1.31-1.46	1.65 1.6
10i. Thickness of gravel—*in*	10	18	10 12	18 16
10j. Maximum size of gravel—*in*	¾-1	2½	1 2	1½ 1½
10k. Minimum size of gravel—*in*	⅜	⅜	½ ½	⅜ ½
10l. Type of strainer system	F	C	I I	B D
10m. Rate of wash (rise per min)—*in*	21	24		30
10n. Agitation of sand	surf. wash	no	surf. wash	no
10o. Height (top of sand to trough crest)—*in*	21	27	36 36	24
10p. Clear distance between troughs—*ft*	4.47	4.67	5.08 6.0	3.25
10q. Location of wash gullet	center	center	front center	center
10r. Wash water tank capacity—*1,000 gal*	125	252	80 250	400
10s. Tank bottom above sand surface—*ft*	32.25	26.25	30.33 42.17	27
10t. Percentage wash water (avg)	2.5-3.0	2.5	2.0	1.3
11. Filtered-water reservoir:			(b)	
11a. Open or covered	covered	covered	covered	covered
11b. Capacity—*mil gal*	33 (b)	2 (b)	40	10
12. Nominal capacity of plant—*mgd*	50	32	40	54

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(5)		(6)	(7)		(8)	(9)
City	Baltimore, Md.		Binghamton, N.Y.	Birmingham, Ala.		Buffalo, N.Y.	Cambridge, Mass.
	Old Plant	New Plant		Shades Mountain Filter Plant	Birmingham Station		
1.	1,162,665		98,000	450,000		580,132 (n)	120,740 (n)
2.	966,233		88,000	375,000		575,150	111,120
3.	1915	1928	1902	1872	1939	1926	1923
4.			191s	1890–1910–1929	1947		1932
5.	Gunpowder River		Susquehanna River	Cahaba River	M. Forks Black Warrior	Lake Erie	Hobbs & Stony Br. & Fresh Pond Res.
6.	(c)			(c)	(c)		(c)
6a.	10–800		43	23	17	0–300	3
6b.	35		58	62	10	95	16
6c.	7.0		7.5	7.6	7.3	8.0	6.90
6d.	5		5			0	35
7.							
7a.	solution		dry	solution & dry	solution & dry	dry	dry
7b.	0.8	0.8	0.6	0.61	0.55	0.2–0.9	1.4
7c.	0.4	0.4	0.1	0.36	0.53	0	0
7d.	36	30	5.5			0.8	1
7e.	78	78	6			90	20
7f.	2,800	2,352	36			75	14
7g.	around end		none	over & under		none	none
7h.	baffles	baffles	mechanical	baffle	tangential	pumps	conduit
7i.	3.3	3±	0.5	0.2–2.0		0	0
8.							
8a.	2	2	3	1	3	2	2
8b.	2.5	3.5	0.75	8 @ max. capac.	6	3.0	1.6
8c.	4.3	3.8	2.5	1	1	1.7	1.5
8d.	around end		none	distr. baffling		training walls	perf. board
8e.	5	5	10	5	4	20	4
8f.	open	open	covered	open	open	covered	covered
8g.	317	380±	88, 95, 121	450±		304	137
8h.	232	1 @ 264± 1 @ 250±	23.5, 16.5, 25	450±		202	48
8i.	12.5–17	15±–18±	10.5	12	15	24	16
8j.	13.5	1 @ 8 1 @ 8.6	0.595	200	3.06	20	1.5
9.							
9a.	2.0	0.65		2	2	4.0	1.33
9b.	1.97	1.97		2	2	2.84	1.33
10.							
10a.	125	125	125	45–60	9–12	125	137
10b.	32	28	21	74	12	40	16
10c.	4	4	12 @ 0.9 9 @ 0.92	0.5 1.0	1	4	1.5
10d.	2 @ 13 × 54	26 × 54	9 @ 321 sq ft 12@15 × 21	15-ft diam.	16 × 22	20.25 × 17.25	2 @ 14 × 50	24 × 20
10e.	9	9	6.25			9	9
10f.	24	24	24	30	30	26	27
10g.	0.60±	0.60±	0.62	0.4–0.5	0.4–0.5	0.45	0.51
10h.	1.2±	1.2±	1.60	1.5	1.5	1.5	1.54
10i.	14	18	18	18	18	16	9
10j.	3	3	1½	2	2	2.5	1
10k.	¼	¼	⅜	¼–10 mesh	¼–10 mesh	⅜	⅜
10l.	A	E	9 @ B, 12 Norwood	ABFH	B	B	F
10m.	24±	24±	13.45 (u)	24	24	24	20–27
10n.	no	no	air	no	no	surf. wash	no
10o.	24	30	16	24	24	24	24
10p.	4.85	3.92	7.67			4.25	2.50
10q.	center	side	center (v)	side & front	front	center	front
10r.	248	242	none	200	156	240	175
10s.	22.5	22.5				48	0
10t.	1.8	2.0	4.0	2.5	2.5	2.5	3.6
11.							
11a.	covered	open	covered	covered	covered	covered	(p) covered
11b.	15 (b)	6.4, 19.4	0.6 (b)	3	0.25	18 (b)	4 (b)
12.	128	112	19.25	55	12	160	24

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(10)	(11)	(12)	(13)	
City	Charleston, S.C.	Charleston, W.Va.	Charlotte, N.C.	Chattanooga, Tenn.	
				Plant 1	Plant 2
1. Population supplied (1950)	120,000	104,000	143,000	203,000	
2. Population supplied (1940)	90,000	91,800	105,000	130,000	
3. Date of beginning operation	1903	1906	1925	1887	1952
4. Date of enlargement	1945–46	1918–1948	1938–1947	1913–18–26–31–41	
5. Source of supply	Edisto River & Goose Creek	Elk River	Catawba River	Tennessee River	
6. Average analysis of raw water:			(c)		
6a. Turbidity—*ppm*		37	31	28	
6b. Alkalinity—*ppm*	13	24	15	53	
6c. pH value (avg)	6.7	6.6	7.5	7.4	
6d. Color—*ppm*	104	0	88	4.0	
7. Mixing chamber:					
7a. Type of chemical feed	dry	dry	dry	solution	
7b. Alum added—*gpg*		0.6	0.57	0.95	
7c. Lime added—*gpg*	none	0.1	none	pre.: 0.41	post.: 0.08
7d. Mixing time at nominal capacity—*min*	5.5	0.4	7	15	45
7e. Velocity at nominal capacity—*fpm*	378	275			
7f. Length of travel—*ft*	2,095	100			
7g. Type of baffling	none	none	none		
7h. Type of mixing	low-service pumps	influent pipe	mechanical & air	tangential	
7i. Head loss in mixing—*ft*		pipe friction only	<0.1		
8. Sedimentation basins:					
8a. Number of basins	2	2	8 (two types)	3	4
8b. Settling time at nominal capacity—*hr*	8	3.6	3.5	3.7	5.0
8c. Velocity at nominal capacity—*fpm*	0.5	0.5	0.4	0.8	
8d. Method of baffling	wall between	vert. slots	none	surface inlet	slotted entrance
8e. Turbidity of settled water—*ppm*		3	2.1	8	12
8f. Basins open or covered	open	open	open	open	open
8g. Length of one basin—*ft*	oval shape	120	96.5	175	
8h. Width of one basin—*ft*	oval shape	58	58.5 or 28	2@ 75, 1@ 120	
8i. Depth of one basin—*ft*	11	22	15–17	15	16
8j. Total basin capacity—*mil gal*	10	3	3.75	2@ 1.3, 1@ 2.3	2.1
9. Settled-water conduits:					
9a. Velocity (filter influent main)—*fps*	4.73	4	0.14	3.1–7.2	
9b. Velocity (filter inlets)—*fps*	1.97	1.3	0.22	1.4–2.8	0.8
10. Filters:					
10a. Rate of filtration—*mgd/acre*		104	96.8	125	125
10b. Number of units	22	20	12	26	4
10c. Size of units—*mgd*	1	0.66	6@ 1.33, 6@ 2.74	20@ 1, 6@ 2	2.5
10d. Dimensions of sand bed—*ft*	18.5 × 20.67	10 × 23	6@ 20 × 12, 6@ 28 × 17	6@ 32 × 22, 20@ 16 × 22	
10e. Depth of filter box—*ft*	7	7	9.25	9	17
10f. Thickness of sand—*in.*	27	30	24	30	30
10g. Effective size of sand—*mm*	0.417	0.37	0.45–0.55	0.42	0.45–0.55
10h. Uniformity coefficient of sand	1.20	1.75	1.09–1.37	1.50	1.60 min
10i. Thickness of gravel—*in.*	9.5	14	15	15	none
10j. Maximum size of gravel—*in.*	1.25	2.5	1.0	1	
10k. Minimum size of gravel—*in.*	10 mesh	¼	1.20 mm	¼	
10l. Type of strainer system	F	B	F and I	H in 20; B in 6	carborundum plates
10m. Rate of wash (rise per min)—*in.*	24	12	30	20–30	24
10n. Agitation of sand	none	air	surf. wash	no	no
10o. Height (top of sand to trough crest)—*in.*	24	15	27	18–28	18
10p. Clear distance between troughs—*ft*	3.25	3.75	5.3	3.17–5.83	3.25
10q. Location of wash gullet	front	none	center	20 front, 6 center	
10r. Wash water tank capacity—*1,000 gal*	136.8	none	137	212	300
10s. Tank bottom above sand surface—*ft*			25	6	9
10t. Percentage wash water (avg)	1.0	2.1	1.18	1.69	1.69
11. Filtered-water reservoir:					
11a. Open or covered	open	covered	covered	open	covered
11b. Capacity—*mil gal*	2	0.6 (b)	12	1.3	1.82
12. Nominal capacity of plant—*mgd*	22	19.8	24.6	32	10

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(14)	(15)	(16)		(17)	(18)	(19)	(20)
City	Chicago, Ill.	Cincinnati, Ohio	Cleveland, Ohio		Clifton, N.J.	Columbia, S.C.	Columbus, Ga.	Columbus, Ohio
			Division Plant	Baldwin Plant				
1.	1,400,000	615,880	914,808 (n)		352,000	87,500	79,510	375,901 (n)
2.	not operating	552,000	1,209,000		340,000		53,280	315,000
3.	1947	1907	1918	1925	1902	1906	1914	1908
4.		1935–37			1917	1916–45	1947	1923
5.	Lake Michigan	Ohio River	Lake Erie					Scioto River
6.								(c)
6a.	10	75	13.8	9.8	6	100	30	70
6b.	110	42	93	93	35	19	14	152
6c.	7.9	7.4	8.1	8.2	7.0	7.0	7.2	8.0
6d.	0		0	0	41	5	none	21
7.								
7a.	dry	dry	solution	solution	dry	dry	dry	solution
7b.	0.6	0.86	0.5–2.5	0.5–2.5	1.27	1.2	0.6	1.9
7c.	0.2	0.76	0	0	8.5	0	0.4	9.2 $Ca(OH)_2$
7d.	45	60	39	1	3	10	30	60
7e.		2	21	600–700			10	88
7f.	700	125	820	30 & 325		150	300	2,130
7g.			over & under (g)	over & under		over & under	over & under	over & under
7h.	mechanical	hydraulic-jump	baffles	hydr.-jump	mechanical	baffles	baffles	mech. & baffles
7i.	<0.1		0.4	2.4	none	2	1	3.3 @ 30 mgd
8.								
8a.	3	2	6	4	3	4	3	6
8b.	3.6	4	3.4	4.67	4	0.75	6	6.67
8c.	2±	2	2.8	2.4	3.6		2	2.5
8d.	none		around end	none		none	over & under	around end
8e.	2.6	2	2.2	1.7	2		2	1
8f.	covered	open	covered	covered	open	open	open	open
8g.	500	500	227.5	693	330	150	152	200
8h.	138	175	142	110.25	51	33	53	100
8i.	35	20	16.92–17.25	9.25–18.5	21	10	20	20
8j.	49	26	21	32.8	7	1.485	3.25	15
9.	(o)							(o)
9a.	0.6	2	2.2	2.17	1.97		2.9	2.0
9b.	1.5		2.94	2.04	1.97	2±	1.1	0.65
10.								
10a.	125	125	125	125		125	125	125
10b.	80	40	36	40	42	18	12	18
10c.	4	4	4.16	4.16	1	1	1.5	3
10d.	25.8 × 53.9	32.7 × 50	2@14×50	2@ 14.83×49	24 × 15	18 × 20	12.25 × 21.25	2@ 11.5 × 47
10e.	13.0	10.3	13.58	13.50	8.0	6	10	11
10f.	24.5	27	24–27	25.5	30	30	28	24
10g.	0.65	0.42–0.44	0.48	0.38	0.45–0.55	0.38–0.45	0.45	0.55
10h.	1.40	1.60	1.50	1.70	1.46	1.3–1.5	1.6	1.60
10i.	21	18	22–25	22	18	18	12	18
10j.	1.5–4	1.5–2.5	1.5	1.5	0.75	2	1.5	2.5
10k.	½	torpedo	½	½	0.25	0.25	½	½
10l.	B	B	B	B	I	6@ F, 12@ B	F	10@ A, 8@ B
10m.	20.8–28.8	24	24–29	24–36	14	23	40	24
10n.	surf. wash		no	no	no	no	no	no
10o.	24	27	26–30	30	22	24	28	24
10p.	3.33	14@ 5.58 26@ 6.17	6.0	4.93	3		5.08	6.0
10q.	side	center	center	center	front		center	center
10r.	300	843	460	2@ 212	65	125	150	104
10s.	60	35.6	35	44.1			28	20.17
10t.	1.75	1.8	2.49	2.03	2.5	1.5	1.25	1.0
11.								
11a.	covered	covered	covered	covered	covered	open	1 covered 2 open	covered
11b.	46	28	19.6 (b)	20 (b)	3.25	6	5.75	16 (b)
12.	320	160	150	165	42		18	54

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

City	(21) Corpus Christi, Tex.	(22) Dallas, Tex.	(23) Denver, Colo. (r) Marston Lake Plant	(23) Denver, Colo. (r) Moffet Plant	(24) Des Moines, Iowa	(25) Detroit, Mich. Water Works Pk. Plant	(25) Detroit, Mich. Spring-wells Plant
1. Population supplied (1950)	125,000	434,462 (n)	415,786 (n)		194,523	1,849,568 (n)	
2. Population supplied (1940)	60,000	360,000	370,000		168,281	1,900,000	
3. Date of beginning operation	1890±	1930	1925	1937	1949	1923	1931
4. Date of enlargement	1917–25–35 1937–42–49	1936–40				1928	
5. Souce of supply		Trinity River	South Platte River	Head-water of Colorado River		Detroit River	
6. Average analysis of raw water:		(e)	(e)				
6a. Turbidity—*ppm*	84	180	2–40	10–52	1.02	5–500 (avg 22)	
6b. Alkalinity—*ppm*	156	130	60–85	20–43	243	83	
6c. pH value (avg)	7.5	8.1	8.1	7.5–8.5	7.31	8.1	
6d. Color—*ppm*		0	5–8	0		0	
7. Mixing chamber:			(p)				
7a. Type of chemical feed	dry	dry	dry	dry	dry	dry	dry
7b. Alum added—*gpg*	2.98	1.0	0.1–1.0	0.6–1.3	0.69	0.74	0.6
7c. Lime added—*gpg*	3.77	5.2	0.05–0.50	0.22	13.5	0	0
7d. Mixing time at nominal capacity—*min.*	15	12	20	7.5	35	3	17
7e. Velocity at nominal capacity—*fpm.*	2.5	150	20	8	1.6	80	60
7f. Length of travel—*ft.*	37.5	1800	400	100	56	240	1020
7g. Type of baffling	over & under	around end	around end	none	over & under	around end	around end
7h. Type of mixing	baffles	baffles	baffles	conduit velocity	mechanical	baffles & mechanical pumps	& mechanical & baffles
7i. Head loss in mixing—*ft.*	0.5	1.5	0.5		0.13	1±	
8. Sedimentation basins:							
8a. Number of basins	4	4	1	3	2	2	4
8b. Settling time at nominal capacity—*hr.*	4–10	8	1.0	4.8	3.3	2.0	2.25
8c. Velocity at nominal capacity—*fpm.*	0.66	1.1	12.0	1.2	1.7	4.2	2.5
8d. Method of baffling	over & under	none	none	around end	over & under	around end	slotted entrance
8e. Turbidity of settled water—*ppm*	8	5–25	3–45	5	2.1	11	10–15
8f. Basins open or covered	open	open	open	covered	covered	covered	covered
8g. Length of one basin—*ft.*	400	2 @ 148 2 @ 407	174.33	290	196	505	338.83
8h. Width of one basin—*ft.*	100	2 @ 148 2 @ 166	166.67	90	96	240	135.5
8i. Depth of one basin—*ft.*	16	18	13.42	14.33	12.4–15.3	16	18
8j. Total basin capacity—*mil gal.*	18	16	2.0	10	8	29	25
9. Settled-water conduits:							
9a. Velocity (filter influent main)—*fps.*	3.0	2.9	2.0	2.8	5.6	3.8	0.9
9b. Velocity (filter inlets)—*fps.*	1.5	1.2	1.7	2.4	3.0	1.97	0.88
10. Filters:							
10a. Rate of filtration—*mgd/acre*	63 156	120	156	150	188	160	160
10b. Number of units	18 6	24	14	10	8	80	68
10c. Size of units—*mgd*	2.5	2	4.5+	5	6	4	4
10d. Dimensions of sand bed—*ft.*	17 30 × 30 × 21.5	2 @ 13×28	23 × 54.33	2 @ 19 × 36.67	2 @ 19 × 36.7	32 × 34	27 × 40.33
10e. Depth of filter box—*ft.*	9 9	10.75	11.75	10	9.3	10.5	11
10f. Thickness of sand—*in.*	30	30	48 (coal)	27	22	30	20
10g. Effective size of sand—*mm.*	0.35–0.45	0.40–0.45	0.62	0.48	0.35–0.45	0.45	0.54
10h. Uniformity coefficient of sand	1.50–1.70	1.60	1.78	1.41	1.25–1.50	1.60	1.52
10i. Thickness of gravel—*in.*	18	18	15	24	24	17	18
10j. Maximum size of gravel—*in.*	2.5	2.5	0.75	2.5	1.5–2.5	2	3
10k. Minimum size of gravel—*in.*	$\frac{1}{2}$	$\frac{1}{2}$	1/20	$\frac{1}{4}$	$\frac{1}{4}$–1	$\frac{1}{16}$	$\frac{1}{16}$
10l. Type of strainer system	B & C	B	B	B	B	C	B
10m. Rate of wash (rise per min)—*in.*	24–30	20	25	27	25	26–30	30–36
10n. Agitation of sand	no	no	air	no	surf. wash	no	no
10o. Height (top of sand to trough crest)—*in.*	18	24	48	27	27	30	40
10p. Clear distance between troughs—*ft.*	5.4	5.0	9.33	2.92	5	5.82	4.44
10q. Location of wash gullet	front & center	center	side	center	central	front	front
10r. Wash water tank capacity—*1,000 gal.*	140	0	150	230	130	140	95
10s. Tank bottom above sand surface—*ft.*	26.5		39.5	40	31	29	35.5
10t. Percentage wash water (avg)	2.22	1.5	1.4	1.44	1.8	2.0	1.3
11. Filtered-water reservoir:							
11a. Open or covered	covered	covered	covered	covered	covered	covered	covered
11b. Capacity—*mil gal.*	39.25	5 (b)	6	10	10	55 (b)	40 (b)
12. Nominal capacity of plant—*mgd*	33	48	64	50	48	320	272

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(26)	(27)	(28)		(29)	(30)	(31)
City	Edmonton, Alta.	El Paso, Tex.	Erie, Pa. Chestnut St. Plant	Erie, Pa. West Plant	Evansville, Ind.	Flint, Mich.	Fort Wayne, Ind.
1.	148,861	130,485 (n)	150,000		150,000	163,143 (n)	133,607 (n)
2.	91,723		124,000		100,000	155,000	118,000
3.	1912	1943	1914	1932	1913	1913	1933
4.	1948	1949	1925	1951	1928–38–48	1918–24–39	
5.	North Saskatchewan River		Lake Erie		Ohio River	Flint River	St. Joseph River
6.							(e)
6a.	15–1800	250	1–90	1–90	110	30	77
6b.	80–260	200	92	92	67	220	209
6c.	8.1	8.3	7.6	7.6	7.3	8.0	7.7
6d.	3–360		10–	10–	2	49	40
7.						(p)	(p)
7a.	dry	dry	dry	dry	dry	dry	dry
7b.	1.50	0.6–1.0	0.4	0.4	1.19	1.1	0.47
7c.	7.75	63	0	0	0.59	12.8	13.2
7d.	34	60	10	40	10	2 rapid–60 slow	5 rapid–60 slow
7e.	57		6.5	36	30	36	170 rapid–54 slow
7f.	42		65				40
7g.	over & under		over & under	none	around end	over & under	
7h.	mechanical	mechanical	baffles	mechanical	baffles	air & baffles	mechanical
7i.	0.7	0.3	1.3	0	1.0	0.7	0
8.							
8a.	2 clarifiers 1 settling (o) (j)	4	2	2	5	3	2
8b.	8.5	3–3.5	2.5	6	4	6.0	4
8c.	1		1.2	1	3.0	2@ 2–3, 1@ 3–4	2
8d.	over & under	Sacramento	around end	none	vertical boards	around end	walls
8e.	15	10–15	1–20	1–20	12	10–15	
8f.	open	open	covered	covered	covered	covered	covered
8g.	115C–270S	105.5 & 148	182	164	80	2@ 175, 1@ 220	84
8h.	115C–114S	67 & 93.75	51	78	60	2@ 100, 1@ 140	84
8i.	17	12 & 15	24	22	27+	2@14.5, 1@13.5	12
8j.	7.8		3.33	4.0	5	7	3.6
9.						(o)	(o)
9a.	2.31				0.6	1.0–3.0	1.22
9b.	1.54				1.5	2.0	
10.							
10a.	124	200	125	125	125–130	125	125
10b.	4	6	16	14	24	28	10
10c.	4.5	3.3	2	2	12@ 1, 8@ 1.5, 4@ 3	1	2.5
10d.	2@ 18 × 44	2@ 12 × 24	712.5 sq ft	712.5 sq ft	21 × 25, 16 × 21	16 × 22.5	28 × 24
10e.	9.67	6.33	10	10	8.5, 8.28	7	14
10f.	30	30	27	27	33	27	24
10g.	0.35–0.55	0.50	0.40	0.45	0.45	0.45–0.50	0.45
10h.	1.65	1.65	1.7	1.6	1.35	1.3–1.4	1.2
10i.	12	14	12@ 6, 4@ 18	14	9	12	16
10j.	1.0	1.0	2.5	1	2.5	2	2
10k.	10 mesh	$\frac{1}{4}$	$\frac{3}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	4 mesh	$\frac{1}{4}$
10l.	F	D	H in 4; I in 12	H in 8; F in 6	B	C	B
10m.	21	30	12–15	12–15	24	16	24–40
10n.	no	no	surf. wash	surf. wash	no	no	no
10o.	27	9	33	33	25	16	24
10p.	4.58	4.42	5.5	5.5	4.0 & 4.5	3.17	5.0
10q.	center	center	center	center	front	front	front
10r.	no	250	113	110	75		100
10s.		42	75	38.67	8	34.5	39
10t	1.3	1.0	1.75	1.75	2.8	2.0	1.64
11.							
11a.	2 open 2 covered	covered	covered	covered	covered	covered	covered
11b.	4.8 (b)	1.1	1 (b)	0.875 (b)	1@ 1.8, 1@ 20	10 (b)	20
12.	18	20	32	28	28	28	25

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(32)	(33)	(34)	(35)
City	Fort Worth, Tex.	Gary, Ind.	Grand Rapids, Mich.	Hamilton, Ont.
1. Population supplied (1950)	277,047	133,911	176,515 (n)	200,000
2. Population supplied (1940)	177,662	111,719	164,000	155,000
3. Date of beginning operation	1912	1910±	1912	1933
4. Date of enlargement	1918–23–32–45–52	1952+	1924–30–40	
5. Source of supply	Lake Worth	Lake Michigan @ 36 mgd @ 54 mgd	Lake Michigan	Lake Ontario
6. Average analysis of raw water:				
6a. Turbidity—*ppm.*	21	9	9	6
6b. Alkalinity—*ppm.*	128	130	100	95
6c. pH value (avg)	8.23	7.8	8.0	7.5–8.5
6d. Color—*ppm.*	<5		0	0
7. Mixing chamber:				
7a. Type of chemical feed	dry / dry	dry	solution	dry
7b. Alum added—*gpg.*	0.76 / 0.76	0.7±	0.5	0.5
7c. Lime added—*gpg.*	0.28 / 0.28	0.09±	0	
7d. Mixing time at nominal capacity—*min.*	37.8–50.7 / 35.3	40 26.5	30	1 rapid, 30 slow
7e. Velocity at nominal capacity—*fpm.*	23.6 / 29+		100	60
7f. Length of travel—*ft.*	1196		3000	60
7g. Type of baffling	around end / around end		around end	none
7h. Type of mixing	mechanical / baffles & mechanical	mechanical	baffles	mechanical
7i. Head loss in mixing—*ft.*			1.0	0
8. Sedimentation basins:				
8a. Number of basins	4 / 3	2–2 story	4	2
8b. Settling time at nominal capacity—*hr.*	(1&2) 3 4 / 5, 6 & 7	4.65 3.1	2.5	3
	7.8 4.67 4.11 / 4.45			
8c. Velocity at nominal capacity—*fpm.*	1.43 0.97 2.0 / 0.96	0.92 1.38	1.6	1.2
8d. Method of baffling	around end / diffuser wall	none	around end	none
8e. Turbidity of settled water—*ppm.*	3 / 3		20	2–20
8f. Basins open or covered	open / open	covered	both	covered
8g. Length of one basin—*ft.*	130 111 308 / 255.3	257	118.75	264
8h. Width of one basin—*ft.*	85 67.5 171 / 75	127.25	88.5	111
8i. Depth of one basin—*ft.*	15.8 / 17.25	top 13.4 bottom 15.2	13.5	14.5
8j. Total basin capacity—*mil gal.*	6.802 / 7.413	7	4	6
9. Settled-water conduits:				
9a. Velocity (filter influent main)—*fps.*	3.94	1.4 2.15	2.5	1.3
9b. Velocity (filter inlets)—*fps.*	1.80	1.33 2.05	2.2	1.5
10. Filters:				
10a. Rate of filtration—*mgd/acre*	120	125 186	118	120
10b. Number of units	12 12 4 16	6	20	12
10c. Size of units—*mgd.*	1.25 2.00 2.00 2.00	6 9	2	4
10d. Dimensions of sand bed—*ft.*	2@9 2@13.2 2@13.2 2@13 ×25 ×27.7 ×27.7 ×18	2@ 17.5 × 60.0	20.5 × 42 20.5 × 36	32 × 45
10e. Depth of filter box—*ft.*	7 8.5 8.5 9.0	10.0	8	10
10f. Thickness of sand—*in.*	27	22	30	30
10g. Effective size of sand—*mm.*	0.37 0.46 0.455 0.447	0.50–0.60	0.5	0.44
10h. Uniformity coefficient of sand	1.86 1.60 1.67 1.54	1.25–1.50	1.33	1.5
10i. Thickness of gravel—*in.*	18 8	24	15	20
10j. Maximum size of gravel—*in.*	1.5 1	1.5–2.5	2.5	2.5
10k. Minimum size of gravel—*in.*	⅛ ¼	¼–½	½	⅛
10l. Type of strainer system	B I	B	A in 15, D in 5	B
10m. Rate of wash (rise per min)—*in.*	19–22 22–24 22–24		23	28–38
10n. Agitation of sand	no	surf. wash	no	no
10o. Height (top of sand to trough crest)—*in.*	24 36 36	27	15	42
10p. Clear distance between troughs—*ft.*	5.0 4.92 4.83	5.0	4.0	5.0
10q. Location of wash gullet	center	center	side	center
10r. Wash water tank capacity—*1,000 gal.*	203.73	300	200	202
10s. Tank bottom above sand surface—*ft.*	19.0	59.75	22	32.5
10t. Percentage wash water (avg)	1.44		1.5	2
11. Filtered-water reservoir:				
11a. Open or covered	covered	covered	covered	covered
11b. Capacity—*mil gal.*	10.742	2.7	8.5 (b)	2
12. Nominal capacity of plant—*mgd.*	79.0	36 54	40	48

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(36)	(37)			(38)			(39)	(40)
City	Hammond, Ind.	Indianapolis, Ind.			Kansas City, Kan.			Kansas City, Mo.	Knoxville, Tenn.
1.	110,000		485,000			129,553 (n)		456,622 (n)	122,500
2.	80,000		380,000			121,000		400,000	165,000
3.	1936	1941	1926	1951		1909		1928	1927
4.		1949		1953		1912–27			1945–51
5.	Lake Michigan	Fall Creek	White River 12 mgd	36 mgd		Missouri River		Missouri River	Tennessee River
6.									
6a.	20	26	33	33		100–20,000		100–20,000	29
6b.	135–145	200	244	244		143		159	67
6c.	7.2–7.6	7.77	8.04	8.04		8.2		8.0	7.8
6d.		18	27	27		26		13	8
7.									
7a.	dry	dry	dry	dry		solution		solution	dry
7b.	0.5	1.12	1.75	1.75		1.12		1.20	1.47
7c.		0	0	0		0.49		0.57	0.70
7d.	40	40	40	40		10		30	20
7e.		1.5	1.5	7.5		65			126
7f.	100	60	60	300		650			
7g.	over & under	floctrol	floctrol	floctrol		around end			none
7h.	mechanical	both	both	both		baffles			mech.
7i.	0.2					0.4		3.5	1.5
8.									
8a.	2	8	1	2		3 (a)		5 (a)	2
8b.	4	2.0	2.3	2.0		33		12.5	8
8c.						2.5		0.6	0.4
8d.	none					none		none	diffuser
8e.	5	1.4	1.0—	1.0—		22		33	3.2
8f.	open	covered	open	covered		open		open	open
8g.	120	192	240	311		201		3@ 309, 2@ 254	175
8h.	100	16	80	97		304		3@ 333, 2@ 250	92–189
8i.	20	15	8.5–13	12.3		21–25		17	25
8j.	3.6	28	1.2	5.6		34		52	5
9.									
9a.		1.6	1.2	0.8		3.0		2.5	0.6
9b.		1.3	0.8	1.9		1.0		2.84	3
10.					(d)		(d)		
10a.	125	32	12	36	1912	125	1927	125	125
10b.	8	12	6	6	9		5	24	10
10c.		2.67	2	6	1.37		2.74	4	2.5
10d.	21 × 42	2@ 14.5 × 30.5	34.75 × 29.75	2@ 57.8 × 36.0	16 × 30		2@ 16 × 30	2@ 14 × 50	13.08 × 33.5
10e.	10	11.50	9.1	11.0	10.83		10.83	9.75	10
10f.	30	27	28	25	26		24	27	30
10g.	0.50	0.45	0.40	0.45	0.39		0.43	0.42	0.45
10h.	1.55	1.40	1.42	1.40	1.62		1.51	1.55	1.6
10i.	18	19	24	19	14		18	18	24
10j.	2	1.5	2.5	1.5	2		2.5	2	2.5
10k.	½	½	½	½	½		½	½	½
10l.	B	H	B	I	A		B	B	B
10m.	30	28–30	28–30	28–30	24		24	24	30
10n.	surf. wash	surf. wash	surf. wash	surf. wash	no		no	no	no
10o.	29	24	24	24	27		24	24	26
10p.	5.0	4.7	3.75	4.50	4.79		4.79	4.33	5.2
10q.	side	center	center	center	side		center	center	center
10r.	100	0.150	0.250		100			150	150
10s.	47				40			22	51
10t.	2.2	2.0	2.0	2.0	2.0			1.37	1.72
11.									
11a.	covered	covered	covered	covered	covered			covered	covered
11b.	5	7.5	16		1 (b)			3 (b)	1.25 (b)
12.	20	32	12	36	26			96	25

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(41)	(42)	(43)			(44)
City	Little Rock, Ark.	Los Angeles, Calif.	Louisville, Ky.			Memphis, Tenn.
1. Population supplied (1950)	160,000	3,352,000	450,000			400,000
2. Population supplied (1940)	125,000	2,028,000	400,000			
3. Date of beginning operation	1938	1941	1909			1953
4. Date of enlargement	1948	1948–49	1914	1928	1945	
5. Source of supply		Lower Colorado River	Ohio River			artesian wells
6. Average analysis of raw water:		(o)				
6a. Turbidity—*ppm.*	10	0.2–10.8	5–5000 (avg 185)			0
6b. Alkalinity—*ppm.*	8	116	55			
6c. pH value (avg)	6.9	8.4	6.7–8.0			
6d. Color—*ppm.*	35	0	0			
7. Mixing chamber:						
7a. Type of chemical feed	dry	both	dry			none
7b. Alum added—*gpg.*	0.4	0.6	0.65			
7c. Lime added—*gpg.*	none		3.0 (softening)			
7d. Mixing time at nominal capacity—*min.*		0.4 rapid, 30 slow	50			
7e. Velocity at nominal capacity—*fpm.*		50 rapid, 20 slow	flocculating equip.			
7f. Length of travel—*ft.*		30 rapid, 400–500 slow	60			
7g. Type of baffling		around end	wood curtain			
7h. Type of mixing	mechanical	mechanical	mechanical			
7i. Head loss in mixing—*ft.*			0			
8. Sedimentation basins:						
8a. Number of basins	2	4	6 (a)			none
8b. Settling time at nominal capacity—*hr.*	6	2	4–5			
8c. Velocity at nominal capacity—*fpm.*		0.5–1.05	1.2			
8d. Method of baffling	none	none	none			
8e. Turbidity of settled water—*ppm.*	5		5–25			
8f. Basins open or covered	open	open	open			
8g. Length of one basin—*ft*		200	340			
8h. Width of one basin—*ft.*		200	40			
8i. Depth of one basin—*ft.*		14–17	20			
8j. Total basin capacity—*mil gal.*	4.0	18.6	12			
9. Settled-water conduits:						
9a. Velocity (filter influent main)—*fps.*		1.93	1.2			
9b. Velocity (filter inlets)—*fps*		1.5	2.3			
10. Filters:			1909 (d)	1914 (d)	1928 (d)	
10a. Rate of filtration—*mgd/acre.*		188	36	72	120	
10b. Number of units	8@2, 14@½	24	6	12	8	10
10c. Size of units—*mgd.*		8.3	6	3	6	3
10d. Dimensions of sand bed—*ft.*		2@ 13.3 × 72.5	42 × 50	23 × 47	42 × 50	2@ 12 × 32
10e. Depth of filter box—*ft.*		10	10	10	10	6.96
10f. Thickness of sand—*in.*	30	24	30	26	30	24
10g. Effective size of sand—*mm.*	0.45	0.45–0.50	0.50	0.45	0.55	0.50
10h. Uniformity coefficient of sand	1.6	1.65	1.3	1.3	1.35	<1.7
10i. Thickness of gravel—*in.*	18	24	24	14	24	10
10j. Maximum size of gravel—*in.*	1.5	2.5	2	1.5	2	<¼
10k. Minimum size of gravel—*in.*	½	½	⅛	⅛	⅛	8 mesh
10l. Type of strainer system	4@ B, 4@ I, 14@ H	B	B	B	B	I
10m. Rate of wash (rise per min)—*in.*	20–30	35	24	24	24	30
10n. Agitation of sand		surf. wash		surf. wash		none
10o. Height (top of sand to trough crest)—*in.*	24 & 36	30	26	26	28	30
10p. Clear distance between troughs—*ft.*	4.0	5.0	5.0	5.17	6.5	3.5
10q. Location of wash gullet	center	center	center	front	center	center
10r. Wash water tank capacity—*1,000 gal.*	150	1000	125	125	300	100
10s. Tank bottom above sand surface—*ft.*	65	40	45	45	45	33.79
10t. Percentage wash water (avg)	1.9	1	2.0	2.0	2.0	
11. Filtered-water reservoir:						
11a. Open or covered	covered	open		covered		covered
11b. Capacity—*mil gal.*	11.0	0		20 (b)		43
12. Nominal capacity of plant—*mgd.*	23.0	200	36	72	120	30

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(45)		(46)	(47)			(48)	(49)
City	Miami, Fla.		Milwaukee, Wis.	Minneapolis, Minn.			Mobile, Ala.	Montreal, Que.
	Hialeah Plant	Southwest Supply		Softening Plant	Columbia Hts. Filter Plant	Findley Filter Plant		
1.	345,398		637,392 (n)		521,718		150,000	1,395,400 (n)
2.	215,950		651,000		492,000		120,000	1,400,000
3.	1925	1951	1939	1941	1913	1927	1944	1918
4.	1936–47				1914–20–52–53	1928–52		1927–30–36
5.	wells		Lake Michigan		Mississippi River		Big Creek Lake (Apr. 1952)	St. Lawrence & Ottawa River
6.								
6a.	nil	nil	2	10.3	10.3	10.3	<10	15
6b.	225	200	110	142	142	142	8	65
6c.	7.3	7.4	8.1	7.94	7.94	7.94	6.8	7.5
6d.	70	3	0	61.8	61.8	61.8	50–90	18
7.								↑
7a.	dry	dry	dry	dry	dry	dry	dry	↑
7b.	0	0	0.5	1.10	1.67	1.89	1.2	↑
7c.	11	92	0.11	7.02			0.2	↑
7d.	4 mix, 20 floc.	center column	4 mix, 60 slow	1	3	6	2	↑
7e.	55	of upflow	70	50	49	20		↑
7f.	110	clarifier	74	50	120	118		↑
7g.	over & under	blanket clarifier			over & under			↑
7h.	mech.		conduit & mech.		baffles			↑
7i.	3.5		0	0.5	0.75	0.75	0.75	↓
8.								↑
8a.	3	4	2	12 units	6	4	4	↑
8b.	1.8		4	1.5	2.4	4.6	6	↑
8c.	0.5		2.32		1	3.5	0.54	↑
8d.	none				around end	around end	perf. wall	↑
8e.	5–15	1.1	0.6	26	1.0	1.5		↑
8f.	open	open	covered	open	covered	covered	open	↑
8g.	2@ 109 × 109		300	85.87 diam.	4@ 131.5, 2@ 120.8	192.5	196.5	↑
8h.	1@ 125 dia.	69 dia.	297	85.87 diam.	2@ 193.3 4@ 95.67		70.75	↑
8i.	17	18	27	23.67	16	16	12.25	↑
8j.	4.5	2	11	120 mgd.	7.8	15.6	5	↑
9.	(0)	(0)						↓
9a.	4	2.5	1.37		3.0	1.4	1.45	
9b.	4.2	2.5	1.8		2.3	1.96		2.8
10.								
10a.	187	187	125		125	125		136
10b.	16	8	32		24	20	10	48
10c.	3.75	5	6.25		3.25	4	2	3.75
10d.	2@ 10 × 43.75	24 × 48	2@ 19 × 57		2@ 11.5 × 5	35 × 40	17 × 41	2@ 12 × 50
10e.	11	11	10		10	10	8.75	10
10f.	30	24	27		30	30	30	30
10g.	0.45	0.50	0.51		0.35–0.44	0.4–0.5		0.48
10h.	1.45	1.70	1.32		1.65	1.55		1.65
10i.	34 on B, 12 on F	12	24		17	18	12	15
10j.	1½ on B, 1 on F	1	2.5		3	2.5–2.75		1.5
10k.	10 mesh	6 mesh	¼		¼	¼		¼
10l.	8 on B, 8 on F	F	D		A	B	F	B
10m.	26	36	24		18	18–24	24	16
10n.	surf. wash	surf. wash	surf. wash		no	no	no	air
10o.	25	31	27		24	25	27	17
10p.	5	5	7.125		5	4	4.5	5.0
10q.	center	side	center		center	front	center	center
10r.	100	186	750		125	132	none	248
10s.	22	34	53		35	36		28.75
10t.	1.1		2.2		1.58	1.40	1–1.5	0.5
11.								
11a.	covered	covered	covered		covered	covered	both	covered
11b.	3 (b)	4 (b)	30		45 + 19.5 + 18	16.5 + 15.2	21	30 (b)
12.	60	40	200	120	78	80	20	180

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

City	(50) Nashville, Tenn.	(51) New Orleans, La.			(52) Norfolk, Va.	
					Moores Bridges Plant	Lake Prince Plant
1. Population supplied (1950)	174,307 (n)	570,445 (n)		59,284	213,513 (n)	
2. Population supplied (1940)	242,000	494,537 (n)		16,180	144,332 (n)	
3. Date of beginning operation	1929	1909		1933	1899	1923
4. Date of enlargement	1932	1924–31		1951	1921	
5. Source of supply	Cumberland River	Mississippi River			Four Lakes	L. Prince
6. Average analysis of raw water:					(c)	(c)
6a. Turbidity—*ppm.*	5–590	75–2350 (avg 700)		313.8	45	18.5
6b. Alkalinity—*ppm.*	36–123	93		106.7	41.6	45.8
6c. pH value (avg)	7.5	7.4–8.2		8.01	7.0	7.15
6d. Color—*ppm.*	1	12		14.1	67	50
7. Mixing chamber:	(p)	(d) 1909	(d) 1926–28			
7a. Type of chemical feed	dry	solution	solution	dry	solution & dry	
7b. Alum added—*gpg.*	1.01	0.75 $FeSO_4$	0.75 $FeSO_4$	0.416	1.49	1.56
7c. Lime added—*gpg.*	0.35	4.98	4.98	6.76	0.69	0.49
7d. Mixing time at nominal capacity—*min.*	26	60	60	2.5	1	10
7e. Velocity at nominal capacity—*fpm.*	48	85	84	7.2	60	40
7f. Length of travel—*ft.*	60	5120	5056	18	60	400
7g. Type of baffling	none	over & under (g)	around end	none	over & under	over & under (g)
7h. Type of mixing	mechanical	baffles	baffles	mechanical	baffles	baffles
7i. Head loss in mixing—*ft.*	0.1	1.5	1.0			
8. Sedimentation basins:						(p)
8a. Number of basins	2 (a)	4 (a)	4 (a)	3	2	2
8b. Settling time at nominal capacity—*hr.*	2.1	18	16	0.75	4.5	5.0
8c. Velocity at nominal capacity—*fpm.*	1.5	0.55	0.52		1.5	1.5
8d. Method of baffling	none	around end	around end		slotted wall	around end
8e. Turbidity of settled water—*ppm.*	4	13	13	6	9	6
8f. Basins open or covered	open	open	open	open	open	open
8g. Length of one basin—*ft.*	188	345	380	50	400	230
8h. Width of one basin—*ft.*	75	250	368	33.5	75±	74
8i. Depth of one basin—*ft.*	17.75	13.75	13.75	14.5	12±	10
8j. Total basin capacity—*mil gal.*	3.75	30	47	0.132	3	2.5
9. Settled-water conduits:						
9a. Velocity (filter influent main)—*fps.*	3.3	1.55	1.37	2.35	4.73	1.5–2.5
9b. Velocity (filter inlets)—*fps.*	1.66	2.85	2.85	1.48	1.9	3.0
10. Filters:						
10a. Rate of filtration—*mgd/acre.*	125	122	122		136	125
10b. Number of units	18	10	18	10	22	12
10c. Size of units—*mgd.*	2.33	4	4	1.0	18@0.67,4@1.0	1
10d. Dimensions of sand bed—*ft.*	25 × 33	2@ 13.5 × 53	2@ 13.5 × 53	18 × 20	11.5 × 18.5	16 × 22
10e. Depth of filter box—*ft.*	8.75	12	13	9.92	8	9
10f. Thickness of sand—*in.*	30	30	36	30	30	30
10g. Effective size of sand—*mm.*	0.40	0.33	0.50	0.32–0.55	0.49	0.38
10h. Uniformity coefficient of sand	1.65	1.65	1.3	1.2–1.6	1.59	1.63
10i. Thickness of gravel—*in.*	18	9	7–10	7	15	15
10j. Maximum size of gravel—*in.*	2.5	1/2	2	1	2.5	2.5
10k. Minimum size of gravel—*in.*	10 mesh	1/4	1/4	1/2	1/4	1/4
10l. Type of strainer system	H	A	B	I	B & C	H
10m. Rate of wash (rise per min)—*in.*	24	24	24	24	24	24
10n. Agitation of sand	no	no	no		no	no
10o. Height (top of sand to trough crest)—*in.*	24	33	30	26	26–28	26–28
10p. Clear distance between troughs—*ft.*	5.0	6.62	6.62	4.33	4.17	3.5
10q. Location of wash gullet	center	center	center	center	front	front
10r. Wash water tank capacity—*1,000 gal.*	200	120		none	0	27
10s. Tank bottom above sand surface—*ft.*	63	19.5				15.5
10t. Percentage wash water (avg)	1.75	0.3		2.06	3.38	1.42
11. Filtered-water reservoir:						
11a. Open or covered	covered	covered		covered	open	covered
11b. Capacity—*mil gal.*	1.3 (b)	20		3.5	2.5	3
12. Nominal Capacity of plant—*mgd.*	42	40	72	7.5	16	12

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(53)			(54)	(55)	(56)	(57)	
City	Oakland, Calif.			Oklahoma City, Okla.	Omaha, Neb.	Ottawa, Ont.	Paterson, Passaic & Little Falls, N.J. Dist.	
	San Pablo Plant	San Leandro Plant	Orinda Plant					
1.		384.575 (n)		243.504 (n)	274.488	197.781		
2.		560.000		204.424 (n)	223.844	165.116	550.000	
3.	1921	1927	1935	1906	1923	1932	1902	
4.	1930	1930		1923	1932	1943	1919	
5.		Mokelumne River		North Canadian River	Missouri River	Ottawa River	Passaic River	
6.	(e)	(e)	(e)	(e)			(c)	
6a.	16	13	3	100	1460	10	3	
6b.	102	117	26	150	187	20	31	
6c.	8.1	7.7	7.0	8.0	7.9	7.0	7.2	
6d.	20	5	0	85	5	45	37	
7.	(p)	(p)	(p)	(p)				
7a.	dry	dry	dry	both	both	dry	dry	
7b.	0.66	0.6	0.30	0.36	1.46	2.0	1.5	
7c.	0.3	0.5	0.06	8.0	0.98	0.7	0	
7d.	8	16	12	30	25	40		
7e.	90	120	43	36	8.8	60		
7f.	720		500	1080	1335			
7g.	over & under	none	over & under	over & under	around end	none	no mixing chamber	
7h.	baffles	mech.	baffles	baffles	baffles	spiral flow		
7i.	1.5	0.2	1.0	2.0	2.2	0.5		
8.								
8a.	2	2		4	6	3	2	
8b.	3	2.7		4.0	1	3.33	4.3	
8c.	1.67	1.1		0.83	1.58	1.0	(d) 1902 3±	(d) 1919
8d.	diffuser wall	diffuser wall		slotted inlet	around end	none	around end	none
8e.	2	5		12	3.65	5	2	2
8f.	open	open		open	open	covered	open	covered
8g.	300	200		190	380	301	310	144
8h.	50	50		2@40, 2@50	370	29.67	110	44
8i.	13	13		12	35	29	20	43
8j.	3	2		2.67	80	7.2	5.7	1.8
9.				(o) (j)				
9a.	1.0	0.75	0.3	1.97	2.05	1.2	1.3	0.4
9b.	1.16	1.16	2.03	2.0	2.46	1.5	2.0	2.0
10.								
10a.	108	108	125	120	157	125	120	125
10b.	16	12	16	8	18	12	32	10
10c.	1.5	1.5	1.75	2	4	4.2	1	1
10d.	20.3 × 30	14 × 44	30.3 × 20.3	2@ 12 × 30	2@ 14 × 50	26 × 56	15 × 24	384 sq ft
10e.	11.5	11.5	11	10±	8	6.67	10	12
10f.	30	30	30	24	22	34	30	30
10g.	0.5	0.6	0.54	0.37–0.42	0.47	0.47	0.44–0.47	0.45–0.50
10h.	1.39	1.5	1.58	1.60	1.53	1.39	1.40	1.40
10i.	18	18	22	18	9	20	12	15
10j.	1.75	1.75	1.75	2.5	1.5	1.5	2	3
10k.	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$
10l.	B	B	B	B	D	B	C	F
10m.	30	24	36	24	2.5	30	12	27
10n.	no	no	no	no	no	surf. wash	air	no
10o.	24	30	35	26	23	26	14	24
10p.	5.0	5.0	4.0	4.62	3.42	5.56	3.0	2.1
10q.	side	side	side	center	center	side	front	
10r.	80	80	200	100	300	300	20	
10s.	40.5	39	42.75	40	30.6	14.75	58	
10t.	1.6	2.24	1.91	2.1	0.57	1.56	2.2	
11.				(p)				
11a.		covered		covered	open	covered	covered	
11b.	6 (b)	3 (b)	9 (b)	3.5 (b)	20	7.2	3.5 (b)	
12.	24	18	28	16	72	50.4	42	

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(58)		(59)		
City	Philadelphia, Pa. (r)		Portsmouth, Va.		
	Queen Lane Plant	Belmont Plant	1942 Plant	1950 Plant	1951 Addition
1. Population supplied (1950)	2,071,605 (n)			150,000	
2. Population supplied (1940)	1,931,334 (n)			80,000	
3. Date of beginning operation	1922	1929	1942	1906	1918
4. Date of enlargement	1925–26		1942	1950	1951
5. Source of supply	Schuylkill River			two lakes and a well	
6. Average analysis of raw water:					
6a. Turbidity—*ppm*	74–251			5	
6b. Alkalinity—*ppm*	49			11.5	
6c. pH value (avg)	7.3			6.3	
6d. Color—*ppm*	9			88	
7. Mixing chamber:					
7a. Type of chemical feed	solution			dry	
7b. Alum added—*gpg*	1.28	1.12		1.19	
7c. Lime added—*gpg*	0	0.20		0.23	
7d. Mixing time at nominal capacity—*min.*		30		43	
7e. Velocity at nominal capacity—*fpm.*		4.1	0.008	0.3	
7f. Length of travel—*ft.*		123	50	95	
7g. Type of baffling	no mixing chamber	over & under	over & under	around end	
7h. Type of mixing		baffles	mechanical	baffles	
7i. Head loss in mixing—*ft.*		0.5			
8. Sedimentation basins:					
8a. Number of basins	4 (a)	2 (a)	2	3	
8b. Settling time at nominal capacity—*hr.*	3.8	2.1	6	6	
8c. Velocity at nominal capacity—*fpm.*	1.2	1.2	0.3	0.9	
8d. Method of baffling	submerged wall	none	over & under	over & under	
8e. Turbidity of settled water—*ppm*	4	9	1	1	
8f. Basins open or covered	open	open	open	open	
8g. Length of one basin—*ft.*	290	150–170	162	110	
8h. Width of one basin—*ft.*	80	115–80	50	95	
8i. Depth of one basin—*ft.*	12.25	11.3	12.7	12.7	
8j. Total basin capacity—*mil gal.*	8	3.4	1.5	3.0	
9. Settled-water conduits:					
9a. Velocity (filter influent main)—*fps.*	1.0	0.3	2	2	
9b. Velocity (filter inlets)—*fps.*	2.2	2.3	1.5	1.5	
10. Filters:					
10a. Rate of filtration—*mgd/acre*	95	100	6	13	
10b. Number of units	20	12	4	10	8
10c. Size of units—*mgd.*	2.5	2.5	1.5	0.9	0.5
10d. Dimensions of sand bed—*ft.*	2 @ 14.5 × 40	44 × 25	2 @ 10 × 26	13 × 23	13.33 × 14
10e. Depth of filter box—*ft.*	9	10	7.08	6.17	7.25
10f. Thickness of sand—*in.*	30	28	30	27	27
10g. Effective size of sand—*mm.*	0.40–0.50	0.40–0.55	0.50–0.55	anthracite	anthracite
10h. Uniformity coefficient of sand	1.50–1.70	1.50–1.70	1.65		
10i. Thickness of gravel—*in.*	18	18	12	12	6
10j. Maximum size of gravel—*in.*	3	2	1.5	1	1
10k. Minimum size of gravel—*in.*	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	20 mesh	14 mesh
10l. Type of strainer system	B	E	F	F	I
10m. Rate of wash (rise per min)—*in.*	12–15	24–30	28	16.6	15
10n. Agitation of sand	air	no		surf. wash	
10o. Height (top of sand to trough crest)—*in.*	15	22	33	26	25
10p. Clear distance between troughs—*ft.*	3.5 & 4.1	4.25	3	4.33	4.33
10q. Location of wash gullet	center	side	center	end	end
10r. Wash water tank capacity—*1,000 gal.*	200	59		75	
10s. Tank bottom above and surface—*ft.*	34	19	30	32	31.5
10t. Percentage wash water (avg)	1.0	3		0.13	
11. Filtered-water reservoir:					
11a. Open or covered	covered			covered	
11b. Capacity—*mil gal.*	50	1 (b)		2.5	
12. Nominal capacity of plant—*mgd.*	50	30		25	

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(60)	(61)	(62)	(63)	(64)	(65)
City	Providence, R.I.	Reading, Pa.	Richmond, Va.	Sacramento, Calif.	Saginaw, Mich.	Saint Paul, Minn.
1.	372,000	109,320 (n)	230,310 (n)	139,700	92,900	311,349
2.	335,000	110,568 (n)	225,000	105,958	82,794 (n)	287,736 (n)
3.	1926	1935	1924	1924	1929	1923
4.	1940		1949	1927–1932	1948	1939
5.	Scituates Res.	Lake Ontelaunee	James River	Sacramento River	Lake Huron	Mississippi River
6.	(c)					
6a.	0.2	15	0–2000	36	4	7
6b.	4.9	78	46	42	77	146
6c.	6.2	7.9	7.4–8.1	7.3	8.0	8.1
6d.	11	13	38		5	68
7.	(p)			(p)		(p)
7a.	dry	solution	dry	solution	dry	dry
7b.		1–1.5	2	1.0	0.5	0.8
7c.	0.61		0	0	2.5	7.45
7d.	4.7 (k)	27	10	80	144	70
7e.		19	45	180	1.8	16–60
7f.	one 35-ft. diam. tangential mixing chamber (k)	750	450		256	130
7g.		around end	over & under	none	over & under	over & under
7h.		baffled channels	both	mechanical	baffles & mech.	air, baffles, & mech.
7i.	1.0	0.3–0.4	1.4	1	0.4	0.17
8.						(a)
8a.	2	3	4 (a)	9 (a)	2	2 prim., 2 sec.
8b.	60	6.6	10	0.5 prim., 1.0 sec.	13.6	2 prim., 3.8 sec.
8c.	0.9	0.93	6.0	2 prim., 4 sec.	1.8	3
8d.	around end	perf. wall		none	around end	around end
8e.	0.1	0.5	8	3	2	8
8f.	open	open	open	open	covered	both
8g.	1@ 800, 1@ 1300	200	320	100 prim., 145 sec.	164	220–110
8h.	1@ 550, 1@ 850	100	120	100 prim., 44 sec.	122	110
8i.	1@ 15, 1@ 18	10.5–13.67	10	10 prim., 9 sec.	20.2	15–19
8j.	1@ 43.4, 1@ 111.4	5.4	7.5	10	6.5	11.6
9.						(o)
9a.	1.2	0.93	1.29	1.23		2.2
9b.	1.1	1.93	2.61	0.86		2.48
10.						
10a.	110	125	120	125	2.0	125
10b.	14	6	10	16	12	18
10c.	4.4	3.33	3	4	3.0	3.5
10d.	2@ 16 × 55	41.5 × 28.17	2@ 12.25 × 44	1,440 sq ft	26 × 26	2@ 14.5 × 42.25
10e.	10.5	11	9	13	12.5	11.25
10f.	30	21	26	24–30	30	12@ 27, 6@ 30
10g.	0.42	0.45	0.43	0.42	2.0	0.40
10h.	1.48	1.6	1.45	1.4		1.75
10i.	18	28	16	16	18	16
10j.	2	2.5	2.5	3	2.5	4
10k.	$\frac{1}{8}$	1/10	$\frac{1}{8}$	$\frac{1}{8}$	$\frac{1}{8}$	1/12
10l.	B	B	B	B	B	E in 12, B in 6
10m.	18–27	40	24	30	21	27–36
10n.	no	no	no	no	no	no
10o.	24	22	23	23	24	22
10p.	2.83	4.92	5.12	5.83	2.3	4.50
10q.	center	center	center	center	front	center
10r.	400	225	100	270	125.5	16,000
10s.	23	57	20	26.5		35.5
10t.	0.8	1.0	0.8–1.0	1.6	0.40	2.48
11.	(p)		(p)			
11a.	covered	covered	open	covered	covered	covered
11b.	0 (m)	10	3 (b)	14.5	9.0	1.9 (b)
12.	61.6	20	30	64	24	63.0

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

City	(66) San Diego, Calif.			(67) San Jose, Calif.	
	Alvarado	Lower Otay	Torrey Pines	Mallard Plant	Petaluma Plant
1. Population supplied (1950)	250,000	80,000	20,000	59,600	14,200
2. Population supplied (1940)	130,000	100,000	20,000		
3. Date of beginning operation	1912	1915	1923	1930	1937
4. Date of enlargement	1920–50	1918–27		1949–51	
5. Source of supply				Sacramento River	Lawler Cr. & Reservoir
6. Average analysis of raw water:					
6a. Turbidity—*ppm*	4			1	
6b. Alkalinity—*ppm*	115	110	120	76	100
6c. pH value (avg)	7.8	7.6	7.6	7.6	7.7
6d. Color—*ppm*	0	0	0		
7. Mixing chamber:					
7a. Type of chemical feed	dry & solution	none	none	dry	dry
7b. Alum added—*gpg*				1.75	2.09
7c. Lime added—*gpg*	1.4			0.80	none
7d. Mixing time at nominal capacity—*min.*	46			12	20
7e. Velocity at nominal capacity—*fpm*	14.8			8.8–26 ppm	variable
7f. Length of travel—*ft.*	680				
7g. Type of baffling	around end			none	none
7h. Type of mixing	mechanical			mechanical	mechanical
7i. Head loss in mixing—*ft.*	0.1				
8. Sedimentation basins:					
8a. Number of basins	2	none	none	2	1
8b. Settling time at nominal capacity—*hr.*	2			3	2.75
8c. Velocity at nominal capacity—*fpm*	2.4			1	
8d. Method of baffling	2-story			around end	around end
8e. Turbidity of settled water—*ppm*	2–6			7	
8f. Basins open or covered	open			open	open
8g. Length of one basin—*ft.*	161			90	circular
8h. Width of one basin—*ft.*	170			43	40 dia.
8i. Depth of one basin—*ft.*	upper 7.33 lower 6.17			13	10
8j. Total basin capacity—*mil gal.*	5.36			0.76	0.083
9. Settled-water conduits:					
9a. Velocity (filter influent main)—*fps*	2.27			6.65	
9a. Velocity (filter inlets)—*fps*	2.6			1.65	
10. Filters:					
10a. Rate of filtration—*mgd/acre*	188	pressure 183	pressure 136	1.5	0.25
10b. Number of units	8	26	7	8	3
10c. Size of units—*mgd*	11.0	0.67	0.5	1.5	0.25
10d. Dimensions of sand bed—*ft.*	2@13.3 × 72.5	horiz. tank 8 × 20	horiz. tank 8 × 20	25 × 14	circular 11′ dia.
10e. Depth of filter box—*ft.*	13	6	6	10	10
10f. Thickness of sand—*in.*	fine coarse 20 4	30	30	28	30
10g. Effective size of sand—*mm*	0.45 1.0	0.5	0.5	0.5	0.5
10h. Uniformity coefficient of sand	1.65 2.0	1.65	1.65	1.5	1.5
10i. Thickness of gravel—*in.*	24	27	27	16	18
10j. Maximum size of gravel—*in.*	2.5	2.5	2.5	2	2
10k. Minimum size of gravel—*in.*	½	¼	¼	⅛	¼
10l. Type of strainer system	B, vit. clay	B, ci & brass	B, ci & brass	B	B
10m. Rate of wash (rise per min)—*in.*	30	20	20	24	24
10n. Agitation of sand	surf. wash	none	none	surf. wash	none
10o. Height (top of sand to trough crest)—*in.*	24	15	15	24	24
10p. Clear distance between troughs—*ft.*	4.92	single tr.	single tr.	6.25	
10q. Location of wash gullet	center			side	center
10r. Wash water tank capacity—*1,000 gal.*	754	pump	pump	none	15.0
10s. Tank bottom above sand surface—*ft.*					30
10t. Percentage wash water (avg)	1.8	3.1	3.1	2	3
11. Filtered-water reservoir:					
11a. Open or covered	covered	none	open	covered	none
11b. Capacity—*mil gal.*	20.0		0.5	0.5	none
12. Nominal capacity of plant—*mgd*	66	16	3	12	0.75

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(68)	(69)			(70)	(71)
City	Savannah, Ga.	St. Louis, Mo.			Tampa, Fla.	Toledo, Ohio
		Chain of Rocks Plant	Howard Bend Plant			
1.	102,000		366,408		124,681 (n)	375,000
2.	105,000	490,000	223,500		108,391 (n)	350,000
3.	1948	1915	1931	1903	1926	1942
4.				1926–48	1948	
5.		Mississippi River	Missouri River		Hillsborough River	Lake Erie
6.						
6a.	25	1430	1400		10	53
6b.	22	132	131		79	96
6c.	6.8	7.95	7.8		7.1	8.0
6d.	25	25	16		100	16
7.						
7a.	dry & solution	solution	dry		dry	dry
7b.	1.1	1.2			5.4	1.125
7c.	0.7	4.7	6.56		2.3	0.502
7d.	45	45	41	28–55	35	40
7e.	1.0	106			1.4	1.5
7f.	300	4760		954	48	234
7g.	slotted	around end	around end	around end	over & under	around end
7h.	mechanical	baffles	baffles & mechanical		baffle	mechanical
7i.		2.5	0	1.5		
8.						
8a.	4	6 (a)	6	3	4	4
8b.	4	36	1.5	5.1–8.4	3.8	2.8
8c.	1	1.5	1.7	1.3–1.8	0.9	1.5
8d.	slotted	none	vertical		over & under	
8e.	1.0	9	273	9.7		2.7
8f.	open	open	open	open	open	covered
8g.	300	400	150	533–653	108	280
8h.	80	670	37.7	109–122	38	85
8i.	15	15.9–23	15.6	9.8–15.5	18	16
8j.	1	240	3.83	16.2	4.25	10
9.		(j)				
9a.	2	1.36–2.53	0.7	1.14	2.03	1.2
9b.	7	2.87	1.6	1.2	1.68	1.26
10.						
10a.	35	125				94.3
10b.	7	40	16	6	12	20
10c.	5	4	2.25	4.34	1.75	4.0
10d.	18 × 50	2@ 14 × 50	12 × 44	20 × 50	2@ 10 × 30	2@ 49.75 × 14
10e.	8	10.5	9	9.5	10	12.83
10f.	30	30	27		30	22
10g.		0.40	0.56		0.55–0.65	0.42
10h.		1.65	1.1		1.70	1.6
10i.		12	12		14	18
10j.		2	1		1.5	1.5
10k.		3/16	1/8		10 mesh	1/4
10l.	F	A	F		B,D	F
10m.	23	24	38		28–30	20
10n.		no	with hose		none	surf. wash
10o.	24	24	30	30	27	35
10p.		4.42	6		4.25	3.5
10q.		center	center	side	center	center
10r.	none	2@ 170	160		200	1000
10s.		35	26		25.5	73
10t.	2	1.67	1.4		1.4	2.43
11.						
11a.	covered	covered	covered		covered	covered
11b.	3	74 (b)	1	105	2.5	35
12.	22	160	60		21	80

* For explanation of letter symbols, *see* Key, p 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

City	(72) Toronto, Ont. Island Plant	(72) Toronto, Ont. Victoria Park Plant	(73) Trenton, N.J.	(74) Tulsa, Okla.
1. Population supplied (1950)	838,520		128,009 (n)	213,000
2. Population supplied (1940)	648,100		124,697 (n)	165,000
3. Date of beginning operation	1916	1941	1914	1929
4. Date of enlargement				1941
5. Source of supply	Lake Ontario		Delaware River	Spavinaw Lake
6. Average analysis of raw water:				
6a. Turbidity—*ppm.*	1–200	1–200	0–1000	4.9
6b. Alkalinity—*ppm.*	96	96	60	83.1
6c. pH value (avg)	7.5–8.4	7.5–8.4	7.5	8.36
6d. Color—*ppm.*	0	0	10–100	15
7. Mixing chamber:				
7a. Type of chemical feed	solution	dry	dry	dry
7b. Alum added—*gpg.*	0.4	0.25	0.7–1.2	0.14
7c. Lime added—*gpg.*	0	0	0	0
7d. Mixing time at nominal capacity—*min.*	2±	40	60	20
7e. Velocity at nominal capacity—*fpm.*	78–630	179	60–80	2.96
7f. Length of travel—*ft.*	417 in pipe		55	
7g. Type of baffling	none	none	down baffles	none
7h. Type of mixing	pump & conduit	spiral flow	mechanical	
7i. Head loss in mixing—*ft.*	0	0.5	0	0.25
8. Sedimentation basins:	↑			
8a. Number of basins		3	2 (a)	3
8b. Settling time at nominal capacity—*hr.*		2.83	3.0	2.7
8c. Velocity at nominal capacity—*fpm.*		1.8	1±	0.84
8d. Method of baffling		none	vert. baffles	
8e. Turbidity of settled water—*ppm.*				2
8f. Basins open or covered		covered	covered	open
8g. Length of one basin—*ft.*		310	209	151
8h. Width of one basin—*ft.*		88	62	102
8i. Depth of one basin—*ft.*		22	19.5	14
8j. Total basin capacity—*mil gal.*		13.5	3.6	4.5
9. Settled-water conduits:	↓			
9a. Velocity (filter influent main)—*fps.*	2.6	3.0	1.0	1.94
9b. Velocity (filter inlets)—*fps.*	1.26–2.36	2.0	1.5	1.97
10. Filters:				
10a. Rate of filtration—*mgd/acre*	180	126	125	
10b. Number of units	10	20	16	10
10c. Size of units—*mgd.*	7.2	6	1.875	4
10d. Dimensions of sand bed—*ft.*	1745 sq ft	2@ 15.5 × 68	2@ 13.5 × 24	14 × 50
10e. Depth of filter box—*ft.*	14	9.3	7	9.5
10f. Thickness of sand—*in.*	108 (f)	26	30	24
10g. Effective size of sand—*mm.*	0.40 (f)	0.55	0.52	0.35–0.50
10h. Uniformity coefficient of sand	2.0 (f)	1.5	1.61	1.6–1.7
10i. Thickness of gravel—*in.*	9	20	12	13½–18
10j. Maximum size of gravel—*in.*	¾ (f)	3	¾	1½–2½
10k. Minimum size of gravel—*in.*	½ (f)	1/20	½	$\frac{1}{8}$
10l. Type of strainer system	B	B	C	6@ B, 4@ D
10m. Rate of wash (rise per min)—*in.*	16	24	19	30
10n. Agitation of sand	no	no	air	none
10o. Height (top of sand to trough crest)—*in.*	51	24	12	26
10p. Clear distance between troughs—*ft.*	special circular gutters	4.63	4.50	5.25
10q. Location of wash gullet		center	center	side
10r. Wash water tank capacity—*1,000 gal.*	240	84	345	120
10s. Tank bottom above sand surface—*ft.*	50.5	27.5	15	11.5
10t. Percentage wash water (avg)	1.37–2.10		3.7	1.0
11. Filtered-water reservoir:				
11a. Open or covered	covered	covered	open	covered
11b. Capacity—*mil gal.*	5.4 (b)	14.4	1.2	17.5
12. Nominal capacity of plant—*mgd.*	72	120	30	40

* For explanation of letter symbols, *see* Key, p. 833.

TABLE 1 (*contd.*)—*Filtration Plants, 1952**

	(75)	(76)	(77)	(78)	(79)
City	Washington, D.C. (r) Delecarlia Plant	Wichita, Kan.	Wilmington, Del. (r)	Windsor, Ont.	Youngstown, Ohio
1.	1,049,000	200,000	110,356 (n)	150,000	220,000
2.	720,131	115,000	112,504 (n)	117,000	200,000
3.	1927	1940	1917	1926	1932
4.	1949	1947–48	1926–39	1952	
5.	Potomac River	wells	Brandywine Creek	Detroit River	Meander Creek Reservoir
6.			(e)		
6a.	80	20	30	45	7
6b.	62	206	41	85	48
6c.	7.7	7.2	7.2–8.6	7.9	7.6
6d.	0	0	26	none	22
7.					
7a.	solution	dry	solution	dry	dry
7b.	1.29		0.9	0.3–5.0	0.5
7c.	0.64	4.9	0	nil	3.5
7d.	21–48	20	4.2	33	27
7e.	73	5	36	2.26	53
7f.	74–108	164	150	75	
7g.	around end & under	down baffles mech.	around end	over & under	
7h.	both	mech.	baffles	mechanical	mech. & baffles
7i.	0.1–0.5	1	0.1	<0.5	
8.					
8a.	5	3	2 (a)	2	4
			(d) (d)		
8b.	3–5.3	2.5	1917 2.4 1926	1.5	2
8c.	2–3.5	1.1–2.4	1.2 2.3	2.26	
8d.	over & under	none	around end none		
8e.	2.1–4.0	10–20	11 11	10	1—
8f.	open	open	covered open	covered	2 open, 2 covered
8g.	327–424	110–160	157 160	265	90
8h.	138–148	110–160	98 55	50	90
8i.	12–17	14	9 15	20	13
8j.	31.4	5	2	3.5	3.5
9.					
9a.	1–1.4	2.0	0.8	1.05	3.2
9b.	2	1.0	2.7	21.0	1.3
10.					
10a.	125	250	118	125	125 winter 150 summer
10b.	46	6	10	20	10
10c.	4	8	2	2.1	4
10d.	27 × 53	2@ 14 × 50	2@ 10.25 × 36	700 sq ft	2@ 57.25 × 12.25
10e.	11–11.45	11	10	9.5	10
10f.	16	36	24	30	27
10g.	0.47	0.52–0.58	0.45		0.5
10h.	1.39–1.46	1.4	1.6		1.5
10i.	18	21	18	18	12
10j.	3	2.5	2.5	1.5	1
10k.	$\frac{1}{2}$	8 mesh	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$
10l.	20@ E, 20@ B	B	B	C	Roberts
10m.	27	24	36	18	28–40
10n.	manual & surf. wash	surf. wash	no	surf. wash	surf. wash
10o.	30–33	28	24	32	27
10p.	3.33	4.5	4.0	5.0	
10q.	side	center	center	end	center
10r.	200	200	100	pumps	180
10s.	46.5	23	35.5		38
10t.	2.29–2.63	1.5	1.97	4.5	3
11.					
11.	covered	covered	covered	covered	covered
11b.	60	10.8	0.6 (b)	12.6	1.1
12.	80	48	20	48	40

* For explanation of letter symbols, see Key, p. 833.

TABLE 2

*Large US and Canadian Cities With Slow Sand Filtration Plants, 1952**

City	Plant Size (*acres*)	Capacity (*mgd*)	Date Built
Albany, N.Y.†	5.6	20	1899
Denver, Colo.	10.0	30	1903
Hartford, Conn.	5.5	25	1921–22
Indianapolis, Ind.–White River	2.4	18	1903
Lowell, Mass.‡		8	1914
New Haven, Conn.	4	12	1906
Philadelphia, Pa.			
Lower Roxborough	2.7	12	1898
Upper Roxborough	5.6	24	1900
Belmont	13.2	50	1904
Queen Land	15	60	1912–15
Torresdale	48.8	200	1905–07
Pittsburgh, Pa.	56	200	1908
Reading, Pa.			
Antietam		5.5	1905
Bernhart		4.5	1911
Springfield, Mass.	6.2	20	1909–24
Toronto, Ont.			
Island Filtration Plant	9.6	58	1911
Washington, D.C.§	29	100	1905
Wilmington, Del.	2	12	1907–10
Yonkers, N.Y.	4	20	1902–08–14

* Only cities of over 100,000 population are included.
† Not operating, but held in reserve.
‡ For iron removal.
§ On low-service area (capacity upgraded from 75 mgd in 1940).

TABLE 3—*Large US and Canadian Cities Not Having Filtration Plants, 1952**

City	Remarks
Baton Rouge, La.	
Boston, Mass.	Impounded mountain water supply
Bridgeport, Conn.	Sodium hexametaphosphate and chlorine applied after long period of storage
Camden, N.J.	Artesian wells
Canton, Ohio	Ground water, chlorination, and fluoridation used
Dayton, Ohio	Well supply
Des Moines, Iowa	Infiltration gallery
Duluth, Minn.	Prechlorination, retention, post-ammoniation, and postchlorination
Elizabeth, N.J.	Water furnished by Newark, N.J., Elizabethtown Water Cons., and Plainfield Union Water Company
Fall River, Mass.	Water impounded in natural lake, chlorination
Fresno, Calif.	Well water
Gary, Ind.	Lake Michigan water (a new filter plant is scheduled for completion in 1953)
Halifax, N.S.	Surface lakes, chlorination
Houston, Tex.	Deep artesian wells (50-mgd water filtration plant under construction)
Jackson, Miss.	Deep wells, chlorination
Jacksonville, Fla.	Artesian wells
Jersey City, N.J.	Surface supply
Los Angeles, Calif.	Impounded mountain water supply
Lynn, Mass.	Surface water
Memphis, Tenn.	Iron removal filters only until 1953
Montgomery, Ala.	Artesian wells, no treatment
Newark, N.J.	Impounded gravity supply
New Bedford, Mass.	Impounded supply from protected watershed
New Haven, Conn.	Surface supplies with chlorination except one sand-filtered supply
New York, N.Y.	Impounded mountain water supply
Peoria, Ill.	Underground supply
Portland, Ore.	Surface supply, chlorinated
Quebec, Que.	Chlorination only
Rochester, N.Y.	Lake supply with chlorination (a filter plant is under construction)
Rockford, Ill.	Deep wells and chlorination
Salt Lake City, Utah	Impounded mountain water supply
San Antonio, Tex.	Deep well supply
San Bernardino, Calif.	Deep well supply
San Francisco, Calif.	Impounded mountain water supply
Scranton, Pa.	Impounded supply (small 6-mgd pressure filter plant on reserve supply)
Seattle, Wash.	Impounded mountain water supply
Somerville, Mass.	Supplied from Boston Metropolitan Water System
South Bend, Ind.	Artesian wells
Spokane, Wash.	Underground supply, chlorinated
Springfield, Mass.	Impounded mountain water supply
Syracuse, N.Y.	Impounded gravity supply
Tacoma, Wash.	Mountain water supply, chlorinated
Utica, N.Y.	Impounded surface supply, pressure filtration
Vancouver, B.C.	Impounded mountain supply, chlorination
Waterbury, Conn.	Impounded surface supply
Winnipeg, Man.	Lake supply
Worcester, Mass.	Impounded supply
Madison, Wis.	Wells

* Only cities of over 100,000 population are included.

Trench Excavation and Backfilling

George F. Sowers

A paper presented on Mar. 20, 1956, at the Southeastern Section Meeting, Augusta, Ga., by George F. Sowers, Prof. of Civ. Eng., Georgia Inst. of Technology, Atlanta, Ga.; Cons. Soils and Foundation Engr., Law-Barrow-Agee Labs., Atlanta, Ga.

A NEW water pipe was being installed in a southern community next to an existing main. A trench was dug with steeply sloping sides, but little bracing was used, for the soil appeared to be quite stable. During the pipelaying, however, the trench caved in and trapped several workmen. Although the men were not killed by the cave-in itself, the old water main burst and they were drowned in the rising water.

This is not an isolated case. The newspapers carry frequent accounts of cave-ins, blowouts, and similar failures which occur in all parts of the United States. In the city of Atlanta, Ga., alone, fifteen men were killed and 24 injured in excavation failures in 1950–1956. Nor are such failures limited to excavations; on the contrary, trouble often occurs long after the work is complete because of the subsidence of a poorly compacted backfill.

Excavation, particularly for water supply conduits, is not a new art. The early civilizations of the Near East produced remarkable examples of deep excavations and tunneling. Unfortunately, the art did not progress greatly until recently. Many trenches today are excavated and backfilled by the same methods used by the ancients.

During the last 30 years, however, there have been many significant advances. First, machines have largely replaced hand labor in both excavation and backfilling. Second, the development of the science of soil mechanics has brought to light the reasons for many present difficulties and has provided methods to cope with them. It is the purpose of this paper to point out what has been learned about the science of excavation and to describe what the engineer can do to make use of these developments in preventing trouble and in obtaining a better job at a lower cost.

Mechanics of Soil Excavation

The procedure employed in making an excavation depends upon the depth of the trench and the nature of the soil. Trenches up to 15 ft deep in firm soils may be excavated by machines to their full depth in a single operation. In soft soils, particularly with deep trenches, the work is done in several stages. First, the trench is made as deep as possible by machine, just as is done in the shallow trench. The limit of depth is determined by the equipment and by the ability of the soil to stand without collapsing. At this point, bracing may be installed to support the soil, after which excavation is resumed, either by hand or by some mechanical device—such as a clamshell—which will fit between the supports.

Before an excavation is made, any small part or element of the soil mass is acted upon by stresses as shown in

Fig. 1*a*. The vertical stress is provided by the weight of the overburden, and the horizontal supporting stress is provided by the surrounding soil.

If a cut is made immediately adjacent to this soil element, the horizontal supporting stress is removed. The block tends to bulge outward in the horizontal direction and to shorten in the vertical direction (Fig. 1*b*). At the bottom of the cut the vertical stress due to the overburden is the greatest, and large amounts of shortening and bulging occur. At the top of the cut the amounts of the shortening and the bulging are much less. These deformations cause the face of the cut to bow outward at the bottom and pull inward at the top, so that the ground surface adjacent to the cut subsides (Fig. 1*c*).

Horizontal tensile stresses develop in the upper part of the soil mass adjacent to the cut. These and the unbalanced compressive stresses in the lower part of the cut face are resisted by the soil strength. So long as these stresses are weaker than the soil strength the cut will be stable. If the stresses should increase or the soil strength decrease sufficiently, the walls of the cut will collapse. Failure starts with the formation of a tension crack at the ground surface parallel to the face of the cut and back from the face a distance of half to two-thirds the height of the cut, as shown in Fig. 2*a*. The crack becomes rapidly deeper until the soil in front of it slides down and outward into the trench (Fig. 2*b*). Occasionally, instead of sliding down, the soil in front of the crack topples over into the trench (Fig. 2*c*).

The bottom of the excavation also undergoes drastic changes in stress. Before an excavation is made, the soil is subjected to a vertical compressive stress equal to the soil weight. Below the trench bottom that stress is reduced by the weight of the soil excavated. The resulting unbalanced stress tends to squeeze the bottom upward—a phenomenon known as heave (Fig. 3*a*). The tendency to heave is resisted by the soil strength; none takes place unless the unbalanced stress exceeds the soil strength.

A third stress change occurs when a trench which has been excavated below the ground water level is unwatered (or drained). The unbalanced head creates an upward (and horizontal) flow, as shown in Fig. 3*b*. In cohesionless soils, such as sand and silt, the upward flow tends to produce a quick (or unstable) condition and sand boiling in the excavation bottom. This tendency is resisted by the weight of the soil. When the water pressure at any point exceeds the weight of the soil above, a quick condition and boiling result.

Causes of Trench Instability

Because failure is caused by stresses which exceed the strength or weight of the soil, anything which increases the soil stress or decreases its strength could lead to failure. The principal conditions which lead to increased soil stresses are the following:

1. An increase in the cut depth (This increases the load on the soil in the bottom face of the cut, increases the soil tension at the top, and increases the unbalance in the bottom.)
2. Saturation of the soil (This increases the weight of the soil.)
3. Water pressure in cracks in the soil (This tends to open the cracks and increase the horizontal stresses.)
4. The weight of excavated soil and machinery on the ground surface adjacent to the cut

5. Shock and vibration caused by heavy machinery and blasting.

Decreases in the soil strength may result from the following:

1. Prolonged stress, particularly tension (This causes soil to deform plastically until it finally breaks.)
2. Excess water pressure or upward flow in sandy soils
3. The swelling of some clays through water absorption
4. The drying-out of damp cohesionless soils such as fine sands (This destroys their capillary forces and apparent cohesion.)
5. The breakdown of loose, unstable arrangements of soil grains.

Most troubles in trench excavations are the result of a combination of these factors. Sometimes failure comes suddenly; in most instances, however, it develops slowly, with ample signs of what is to come. The first sign of trouble in the walls of the cut is the subsidence of the adjacent ground surface. Continuing subsidence is especially serious. The second sign is the formation of tension cracks parallel to the trench. The final indication is the spalling of small pieces of soil from the cut face.

Bottom heave ordinarily occurs slowly and often goes unnoticed. Subsidence of the ground surface in braced cuts and the excavation of excessive volumes of soil are both indications of this form of trouble.

A quick bottom condition ordinarily develops rather suddenly during unwatering. Occasionally small boils will appear in isolated points and give warning of the instability. In other instances the bottom will heave slightly and then boils which cover a large part of the bottom will form.

Wet weather and periods of melting snow should be considered potential sources of trouble. It is significant that most failures occur during late winter and early spring, when the soils are wet and the water table high.

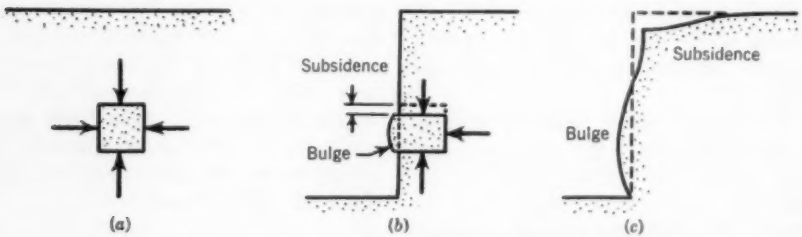

Fig. 1. Stresses and Deformation in an Open Cut or Trench

Section (a) *shows directions of stress in an unsupported soil mass. Section* (b) *is an unsupported cut, showing unbalanced stress in soil. Section* (c) *is a profile of a cut demonstrating typical subsidence and bulging.*

Preventive Measures

Prevention of excavation trouble involves control of one or more of the factors producing stress increase or strength loss.

Limiting the loads applied to the strip of ground adjacent to the excavation can help prevent a serious increase in soil stress. The excavated soil should be kept back from the edge a distance equal to half the trench depth

or more. Likewise, loaded trucks and other equipment which need not be operated close to the trench should be kept away. This latter precaution will also reduce the stresses which may be produced by the vibration and shock of equipment operation—a very real danger in loose sands and silts.

The best method of preventing stress increases in the soil is to provide support by adequate bracing. Bracing will reduce the bulging in the deeper part of the ditch and will prevent the formation of tension cracks at the top. Three different systems are in common use. The simplest consists merely of horizontal struts which bear against vertical planks placed at intervals along the trench, as shown in Fig. 4*a*. When the soil is fairly firm and the depth limited, simple strutting can be very effective. When continuous support is needed in soft clays or loose water-bearing sands, a sheeting system may be necessary (Fig. 4*b*). This consists of vertical sheets of wood or steel, horizontal wales, and struts. When water control is not necessary and where underground obstructions make it impossible to drive continuous sheeting, a third system (Fig. 4*c*) may be employed. Vertical H beams are driven about 6 ft apart. Horizontal boards are placed behind the flanges as the excavation progresses to support the soil. The H beams are supported by a system of wales and struts similar to that used in the sheeting system. The knowledge of the earth pressure required for the design of bracing systems has advanced rapidly in recent years. Methods for analysis and design are presented in a number of text books on soil mechanics (*1–3*).

Normally a bracing system need extend only the depth of the trench. In cases where an unstable bottom is involved, driving sheeting below the bottom of the ditch can reduce the instability. When the trouble is a squeezing of clay, the friction developed along the sheeting effectively reduces the upward squeeze. When upward water flow is causing a quick condition, driving the sheeting deeper often reduces or even cuts off the flow.

Drainage will reduce the water pressure in cohesionless soils and prevent unstable cut faces and quick bottoms. Often drainage will make it possible to reduce the amount of bracing required in deep trenches or even to omit

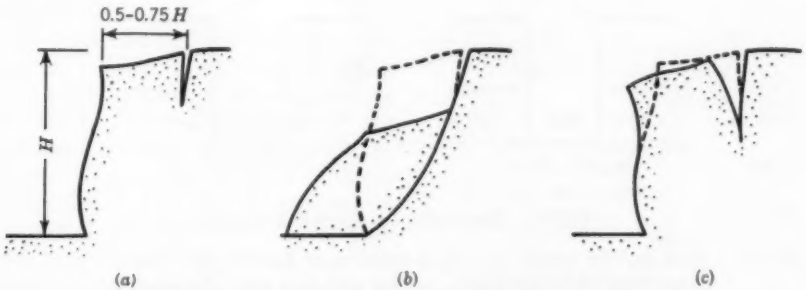

Fig. 2. Methods of Failure in a Trench or Unbraced Cut

Section (a) *shows the formation of tension cracks, which occur a distance half to two-thirds the depth of the cut back from the face of the cut. Section* (b) *shows the sliding of soil into a trench, Section* (c) *a toppling movement.*

it altogether. Lowering the water table by drainage may also reduce the tendency of some clays to swell. (Actual drainage takes place in this case through cracks in the clay or sand seams, because no method for draining clays rapidly exists). Unfortunately, drainage may be a mixed blessing in some situations. Prolonged lowering of the water table can cause settlement of nearby structures, particularly if they are founded on highly compressible soils. In some cases, improperly installed drainage in fine sands can remove particles of the soil and cause subsidences. Continued drainage of a sand can cause it to become progressivly drier until even the capillary moisture is lost. Because capillary moisture adds to the strength of fine sands, its loss can bring about sliding in the cut face of a trench.

The simplest form of drainage is a system of sumps. Although sumps are cheap they are not very effective in fine sands and are likely to create unstable bottom conditions. The most effective drainage is provided by well points. These can be installed before excavation starts so all the work can be done under dry conditions. In addition, by draining the water table below the trench bottom, upward flow can be prevented and danger from instability eliminated. Well points with vacuum can accelerate drainage in silty soils and thus be effective more quickly than plain gravity drainage.

Not all the water trouble comes from ground water. Surface water getting into the tension cracks can increase the soil stresses, and water running down the face of the cut (particularly a sloping face) can cause soil erosion and sloughing. Diversion ditches can do much to keep water away from the trench and the excavated soil can be used as a temporary dike to accomplish the same purpose. If the tension cracks are small they can be filled with asphalt to keep water out. Large tension cracks and the cut face can be protected with canvas or roofing paper.

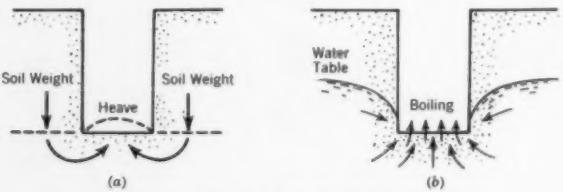

Fig. 3. Instability of Trench Bottoms

Section (a) *shows a typical pattern of bottom squeezing or heave caused by the weight of the adjacent soil. Section* (b) *shows an upward water flow creating a boiling or quick condition of the trench bottom.*

The last preventive of trouble consists of minimizing the time between excavation and backfilling. Although other factors often control the job schedule, the danger of failure is reduced by shortening the length of time the excavation is open.

A prerequisite to any preventive measure is accurate data about the soil conditions along the line of the trench. Ordinarily the data needed include: [1] the depth and thickness of the different soil or rock strata which will be

encountered; [2] the approximate strength of each stratum; and [3] the depth to ground water. Simple auger borings made at intervals of 100–500 ft, depending on the variability of the soil, should be adequate for most situations. When deep trenches or soft soils are encountered, it may be necessary to secure intact samples of the soils and test them in a laboratory to determine what measures will be necessary. Organizations specializing in such investigations can now be found in all parts of the United States and their services usually save much more than they cost.

Backfilling

The purpose of backfilling is threefold: to support and protect the pipe; to support adjoining structures; and to provide a good foundation for the pavement or whatever else might be above. The earliest method of backfilling consisted of merely dumping as much soil as possible into the trench and allowing it to settle for a few weeks. Unfortunately, the soil continued to settle for years and the continued settlement sometimes caused the pipe to move and leak. In sandy regions the soil was washed or settled into the trench with a fire hose. The results were comparable to those produced by dumping.

With heavier and heavier vehicles on roads, the next development was traffic compaction. The trench was dumped full and automobiles and trucks allowed to operate over the soil. From time to time additional soil was added to make up for the compaction. This method produced high densities in the uppermost 1–2 ft of the soil but unfortunately did not affect the deeper soils. As a result, the loose deeper soil would subside slowly and create a cavity beneath the compacted crust. Sooner or later a heavy truck would break through the crust and a hole would result.

In the late 1920's a new concept of compaction developed. It was found that the degree of compaction obtained varies with the amount of moisture in the soils, as shown in Fig. 5. When the moisture exceeds or is less than the critical or optimum amount, the density will be less. The maximum density, as now defined by standard ASTM tests, reflects the best compaction obtainable with a moderate amount of rolling or tamping. The maximum also reflects the quality of the soil itself: the higher the maximum density the better suited the soil is to compaction. In general, a soil compacted to the maximum density will be strong and incompressible and should provide excellent support for heavy traffic. The ratio of the actual soil density to the maximum density is the percentage of compaction. This indicates the quality or degree of compaction and the ability of the soil to carry heavy loads without settlement. Table 1 presents a classification according to soil compaction requirements.

TABLE 1

Backfill Classification According to Soil Compaction Requirements

Class	Type of Backfill	Percentage of Compaction*
1	Supporting extremely heavy loads	100+
2	Subject to surface loads	95–97
3	Not subject to surface loads	90–93
4	Where settlement causes no harm	85–90

* Ratio of actual soil density to maximum density, as established by standard ASTM tests.

Backfilling a pipe trench should be done in four separate steps (*see* Fig. 6). The first consists of bedding the

pipe. The width of the bed should be equal to at least half the pipe diameter and it should be either cut in virgin soil or made of Class 1 fill. It is particularly important that the bedding be the same material throughout each length of pipe. Support by rock at one point and soil at another can cause the pipe to crack and fail. The same holds true, to some extent, for hard soil at one point and soft at another. In case of doubt it is better to excavate the trench deeper than is required and to make a uniform bed of compacted fill throughout. In the second step, fill, compacted to at least Class 2 but preferably Class 1, should be placed around the pipe to a height of two-thirds to three-fourths of its diameter. The greater the degree of compaction of the soil on the sides of the pipe, the less the stress in the pipe developed by vertical loads. The third step is backfilling with Class 3 compaction to a height of 1–1½ times the pipe diameter. This creates a zone of somewhat looser soil above the pipe which can help protect it against excessive vertical loading. The remaining backfill is completed with Class 2 or Class 1 compaction, depending on the surface loads.

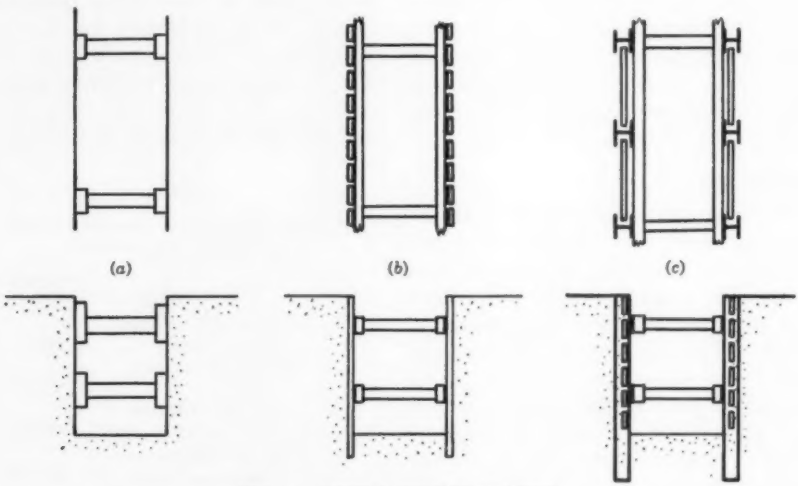

Fig. 4. Trench Bracing Systems

Systems shown in plan (above) and section (below) are: (a) *a simple strut bracing system with horizontal struts bearing against vertical planks at intervals along the trench;* (b) *a sheeting system, used in soft clays or loose water-bearing sands, comprised of vertical sheets of wood or steel, horizontal wales, and struts;* (c) *vertical* H *beams supported by a system of struts and wales, used when water control is unnecessary and where obstructions render the use of continuous sheeting impossible.*

Many different compaction procedures may be employed. The traditional methods of dumping and washing seldom produce compaction as good as Class 4. While these methods are initially cheap, cost studies indicate that their ultimate cost is greater than that of well compacted backfills, because of the increased maintenance. Traffic compaction may produce Class 2 densities in the uppermost 12–18 in.

of backfill, but Class 4 and poorer below.

A number of mechanical devices are available which can produce all classes of compaction, depending on their use. All these devices work most efficiently when the soil moisture is approximately equal to the optimum and all require that the backfill be placed in layers of limited thickness. Hand tampers and pneumatic tampers are both effective in compacting most soils to at least Class 2 densities, provided the layers are no thicker than 4 in. Class 1 compaction can be obtained with thinner layers. A heavy gasoline engine–driven jumping tamper can compact soils in layers up to 12 in. thick (and occasionally more) to Class 2 densities, and layers up to 9 in. thick to Class 1. Several different heavy tamping machines are available which resemble miniature pile drivers. These are mounted on wheels which straddle the trench and they employ tampers which weigh as much as 500 lb and which fall as far as 5 ft. These devices can produce Class 1 compaction to a depth of 1–1½ ft and Class 2 compaction to a depth of 2–3 ft, and are probably the most effective compactors yet developed. Several types of vibrating compactors are available. These are of use only in cohesionless soils such as sands and will produce Class 1 or 2 compaction as deep as 6 in.

Because poorly compacted soil often looks no different from well compacted soil, it is necessary to measure the density of the backfill. This is done by excavating or cutting out a sample of the compacted soil, measuring its in-place volume, weighing it, and determining its water content. The actual or field density is compared to the maximum density to determine the percentage of compaction. The fact that a contractor puts all the soil back in the trench that he excavated is no indication of good compaction. Many soils, including loose cemented sands and hard clays, often exhibit a loose but incompressible grain structure. When this structure is destroyed by excavation, a much greater density becomes necessary to restore the soil to its original incompressibility.

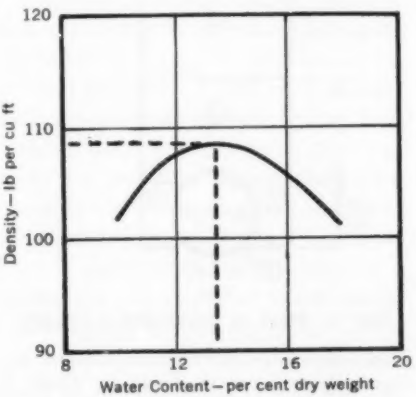

Fig. 5. Typical Water Content–Density Curve

Density values are for dry soil. The vertical broken line represents critical or optimum moisture content for given soil; when the moisture exceeds or is less than this amount, the density of the soil will be less. The maximum density (horizontal broken line), as defined by standard ASTM tests, reflects the best compaction obtainable with a moderate amount of rolling or tamping.

Soil compaction is a complex art. Considerable experience and theoretical knowledge are necessary in order to obtain good compaction economically. Where accurate cost studies are made of compacted and uncompacted soils, the results indicate that the money spent for compaction is more than saved by lowered maintenance.

Summary

Trench excavation and backfilling have come a long way since the early days of hand excavation and dumped backfill. At one time they were considered arts, and the methods used were dictated only by experience and tradition.

Excavation problems are primarily the result of great depth, weak soils, excess water, and external loads. These can be controlled by drainage, bracing, and speedy prosecution of the work.

Backfilling involves an understanding of the maximum density–optimum moisture concept and the compaction requirements of different parts of the trench.

Modern engineering has made it possible to dig pipe trenches deeper, more quickly, and more safely, and to backfill them better. The result has been more trench footage at a lower cost—a primary aim in all construction.

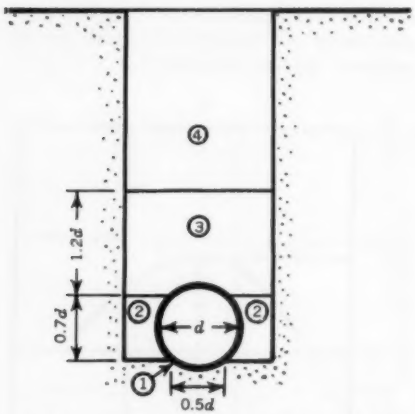

Fig. 6. Steps in Backfilling a Trench

Numerals in circles represent class of fill required, as listed in Table 1. Class 1 includes fills of the highest compaction requirement, Class 4 of the lowest. The d *factors represent depth of various fills in relation to the diameter of the pipe used.*

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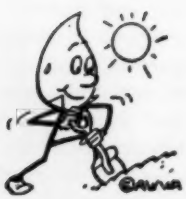

Development and Use of Specifications for Cast-Iron Pressure Pipe and Fittings

Jack W. MacKay

A contribution to the Journal by Jack W. MacKay, Vice-Pres. Sales, American Cast Iron Pipe Co., Birmingham, Ala.

IN recent years, engineers and users of cast-iron pressure pipe centrifugally cast have frequently been confused about the question of which pipe specifications to use. This situation can be attributed primarily to the number of specifications available, and also to applied modern metallurgical science, better testing methods, improved foundry control, increased strength of materials, and improvements in manufacturing methods. Another important factor has been the accumulation of a wealth of technical data, including extensive tests to determine the suitability of pipe under various loadings, and methods of using the results of these tests to determine correct pipe specifications for given operating conditions.

Engineers and purchasers have continued to refer to the older standards because of a history of prior successful use and because of a lack of familiarity with more recent specifications. Uncertainty as to how to specify pipe correctly for particular service conditions has possibly contributed to the generally retarded use of the new specifications. Such neglect may impose a burden on the buyer or result in an unsafe design for the engineer.

A brief review of the older specifications, in chronological order, will define their usefulness and provide the designer with an insight into current standards, which may be used with confidence and economy.

Early Specifications and Standards

Barlow Formula

The Barlow formula, sometimes called the "hoop tension" formula, is the fundamental expression for design of a thin cylinder under internal fluid pressure. It has been used, in whole or in part, as the engineering basis for design formulas in almost all pipe specifications to date. This formula may be stated either:

$$t = \frac{pr}{S}$$

or:

$$t = \frac{pd}{2S}$$

in which t is the thickness of the pipe, in inches; p is the internal pressure, in pounds per square inch; r is the internal radius of the pipe, in inches; d is the inside diameter, in inches; and S is the allowable working stress of the metal, in pounds per square inch.

The formula has been modified in several ways by prominent water sup-

ply engineers to make it applicable to the design of cast-iron pressure pipe. These modifications, expressing the judgments and observations of their various authors, include allowances for water hammer, earth pressures, handling, foundry tolerance, and corrosion.

Both Allen Hazen and Thomas H. Wiggin, who have contributed greatly to the advancement of cast-iron pipe specifications, devised modifications of this fundamental formula providing for the consideration of both the tensile fiber stresses resulting from flattening of the pipe under trench load, and the direct tension due to internal water pressure. In the Hazen and Wiggin modifications, the sum of these two stresses was kept within the assumed working stress, which was obtained by dividing the ultimate strength by a factor of safety.

Antedating the organization of both AWWA and NEWWA is the report of James P. Kirkwood, chief engineer of the Brooklyn (N.Y.) water works. This rare document, published in 1867, is a valuable part of the lore of early pipe design. The report describes studies made by Kirkwood and his associates in designing pipe for installation in the Brooklyn system. In the report the author refers to the following formula, which was used in the pipe design:

$$t = \frac{5pr}{c - p} + x$$

in which t is the thickness of the pipe, in inches; p is the pressure, in pounds per square inch, created by the head of water; r is the internal radius of the pipe, in inches; c is the allowed cohesion of the iron, in pounds; and x is a constant added to account for defective casting, handling, rust, or other factors affecting the life of the pipe.

Remarks following the formula describe the tensile strength of the castings examined as 20,000–22,133 psi. Values for the term x are listed in a separate table, and range from 0.35 in. for 4-in. pipe to 0.24 in. for 36-in.

TABLE 1

Water Hammer Allowances Used in Determining Pipe Wall Thickness

Nominal Diam. of Pipe *in.*	Fanning Formula	Brackett Formula	Fairchild Formula	ASA Standard Allowances†
	Water Hammer Allowance—*psi*			
3–10	100	120	120	120
12–14	100	110	110	110
16–18	100	100	110	100
22	100	90	100	90
24	100	85	95	85
30	100	80	90*	80
36	100	75	85*	75
42	100	70	75*	70
48	100	70	70*	70
54–60	100	70	—	70

* Values adopted for calculating thicknesses of centrifugally cast pipe.
† From Table 1, ASA A21.1.

pipe. As may be seen in the formula itself, a safety factor of 5 was used. Kirkwood describes the formula as a "variant" of the Barlow formula.

The report also uses the terms "Class A" and "Class B" to describe pipe thicknesses. An Appendix includes specifications for cast-iron pipe, with a memorandum concerning the Angus Smith tar dip, a requirement of the standard.

Brackett Formula

The Dexter Brackett or NEWWA formula was one of the most prominent early variations of the Barlow formula. It was probably in use as early as 1885 or 1890 by the Boston water works, of which Mr. Brackett was distribution engineer. This formula is as follows:

$$t = \frac{(p + p')r}{3{,}300} + 0.25$$

in which t is the thickness of the pipe wall, in inches; p is the static pressure, in pounds per square inch; p' is the pressure allowed for water hammer, in pounds per square inch; r is the internal radius of the pipe, in inches; 3,300 is equal to one-fifth the tensile strength of cast iron, taken as 16,500 psi; and 0.25 is an allowance for corrosion and foundry tolerance, in inches. Water hammer allowances for this and the other formulas discussed are shown in Table 1.

In 1895, Brackett became engineer in charge of distribution for the Metropolitan Water Board of Massachusetts, and supervised a standard for that board in which pipe thicknesses were based on his formula. The most outstanding addition to specifications of this era was that of the 2 × 1-in. test bar, which was part of the Metropolitan Water Board standard. The late Alfred D. Flinn and Thomas H. Wiggin assisted in preparing the standard. The 2 × 1-in. test bar was also made a part of the 1902 NEWWA specifications and the 1908 AWWA specifications.

Although the 1902 NEWWA specifications do not provide a formula for pipe thicknesses nor the pressures for which the various classes of pipe, A through K, are recommended, the original Brackett formula was used in computing pipe thicknesses. These specifications covered pipe produced by the pit-casting method and can now be considered obsolete.

AWWA Specifications, 1908

The AWWA specifications of 1908 were the best known and most widely used standard from the time of their adoption until they were superseded by the current AWWA specifications which cover both pit-cast and centrifugally cast pipe.

Although the NEWWA specifications antedated the AWWA specifications by 6 years, it is worth noting here the existence prior to 1902 of two unofficial documents of AWWA. These were gleaned from the record of the 1890 AWWA convention. The first is a report by S. B. Russell of St. Louis, which includes a tabulation of dimensions and weights for classes A, B, and C pipe of 3–48-in. sizes. The weight of Class B pipe was determined from the average weight of pipe used in a large number of American cities. It is assumed that the figures relating to pipe thickness, which are listed for each size only, were similar averages. Class A pipe was 10 per cent lighter than Class B, and Class C pipe was 10 per cent heavier than Class B.

The second document is a report by Thomas W. Yardley of Chicago, which includes sixteen paragraphs of what would now be called specifications for cast-iron tar-coated water pipe. Section 8 of these specifications covers the use of the standard 2 × 1-in. test bar.

From the discussion which followed the presentation of these two reports, it is evident that manufacturers were not then prepared to standardize. It is, however, understood that these documents were used informally until the promulgation of the NEWWA specifications in 1902.

The 1908 AWWA specifications covered bell-and-spigot pipe produced in 12-ft laying lengths by the pit-casting method using stationary sand-lined molds and sand-surfaced cores to form the inside. These specifications are now considered obsolete, except for that part pertaining to the manufacture of bell-and-spigot fittings, which has been rewritten and is now designated AWWA C-100 (*1*). Some bell-and-spigot pipe, for the most part of 30-in. diameter and larger, is still produced by this method, but the current specification for bell-and-spigot pit-cast water pipe is ASA A21.2 (AWWA C102) (*2*).

The AWWA specifications employed a system of class designations applied to specific pipe wall thicknesses in diameters of 4–84 in. inclusive, for a range of hydraulic heads. The more common of these are classes A, B, C, and D for 100-, 200-, 300-, and 400-ft hydraulic heads, or 43, 86, 130, and 173 psi, respectively. The pipe wall thicknesses were calculated for internal pressure only, using a tensile bursting strength of 18,000 psi, plus an arbitrary allowance for corrosion and foundry tolerance. Although the estimated tensile strength was considered conservative, later tests by ASA Sectional Committee A21 showed the actual strengths were generally lower.

The James T. Fanning formula was apparently used in modified form in computing thicknesses for the several pipe classes of the AWWA specifications. The Fanning formula is as follows:

$$t = \frac{(p + 100)d}{0.4S} + 0.333\left(1 - \frac{d}{100}\right)$$

or:

$$t = \frac{5(p + p')r}{18{,}000} + 0.333\left(1 - \frac{d}{100}\right)$$

in which t, p, p' and d are, as before, thickness, static pressure, water hammer, and diameter, respectively. S is the supposed ultimate tensile strength of cast iron in pounds per square inch.

The first term is the fundamental formula with a safety factor of 5, as in the Brackett formula. The second term is an arbitrary allowance which varies from 0.32 in. for 3-in. pipe to 0 for 100-in. pipe. The value of p' was taken as 100 psi for all pipe diameters.

Pipes in sizes of 4–12 in. were produced to two outside diameters for each size but used only Class D fittings. Each size of pipe, 14–24 in., was manufactured in two outside diameters. Classes A and B pipe were made to the A-B or smaller outside diameters and used Class B fittings. The larger outside diameters (C-D patterns) were used for making classes C and D pipe, which utilized Class D fittings.

For pipe of 30 in. and larger, four different outside diameters—designated A, B, C, and D—were used for the respective classes of pipe. Four different classes of fittings of like desig-

nation and with the same pressure ratings as the corresponding classes of pipe were used. The various outside diameters were set in such a manner that the actual inside diameter was always equal to or greater than the nominal size of the pipe.

AWWA Specifications, 1939

In 1939, AWWA adopted the new method of designing cast-iron pressure pipe, ASA A21.1 (AWWA C101) and the pit-cast pipe specification, ASA A21.2, which is based on the new law (*2, 3*). A revision of A21.1 is currently under way.

Federal Specifications

With the emergence of centrifugally cast pipe, higher-strength iron was obtainable and consequently a reduction in pipe wall thickness could be realized for the same internal pressure design conditions. Federal Specifications No. 537, published in 1927, were the first covering the manufacture of centrifugally cast iron pressure pipe.

Subsequently, Federal Specifications WW-P-421, dated July 21, 1931, were published. These specifications have been modified several times. Originally the specifications covered three types of bell-and-spigot pipe produced commercially in laying lengths of 12, 16, 16½, 18, and 20 ft, in sizes of 4–24 in. inclusive, for two operating pressures. Class 150 pipe is for a maximum working water pressure of 150 psi, and Class 250 for 250 psi.

Type I pipe is produced centrifugally in metal molds (deLavaud process); Type II pipe is manufactured centrifugally in sand-lined refractory molds (sand-spun process); and Type III (no longer available commercially) was made in stationary, on-side green sand molds with green sand cores.

The modified Fairchild formula, which was used to determine pipe thicknesses in the federal specifications, was developed by I. J. Fairchild and originally published in 1927. The modified formula is as follows:

$$t = \frac{fd(p + p')}{2S} + \frac{0.28}{d^{0.15}}$$

or:

$$t = \frac{5(p + p')r}{25{,}000} + \frac{0.28}{d^{0.15}}$$

in which f is a factor of safety which, for centrifugally cast pipe, is taken as 5; S is the ultimate bursting or tensile strength of the metal in the pipe wall, assumed to be 25,000 psi; and the other factors are the same values as previously stated.

The first term is the fundamental formula for thin-wall cylinders. The second term is an arbitrary allowance for handling, foundry tolerance, and indeterminate stresses. It works out as 0.23 in. for 4-in. pipe and 0.17 in. for 24-in. pipe.

Pipe of 4 and 6 in. is made to the AWWA outside diameter A; 8-, 10-, and 12-in. pipe is made to the AWWA A-B diameters. Pipe of 14–24-in. size is produced to the AWWA C-D diameters. AWWA Class D fittings are used with all sizes and classes of pipe manufactured *in strict accordance* with federal specifications. If desired, Class 250 short-body bell-and-spigot fittings (ASA A21.10) can be used with 4–12-in. pipe only.

Although the specifications provided that 14–24-in. pipe be made only in Class 150 and Class 250 to the larger diameters for use with AWWA Class D fittings, centrifugal pipe manufacturers immediately adapted the design thicknesses for pipe produced to the

same pressure ratings on the A-B or smaller diameters, to be used with AWWA Class B fittings. Some engineers specified such pipe as "Class 150 pipe designed by the federal specifications formula to be used with Class B fittings." In addition, the formula was used to calculate thicknesses for other operating pressures for pipe made to either the larger or smaller diameters.

Until development of the ASA specifications, the Fairchild formula was used exclusively in calculating thicknesses of cast-iron pressure pipes centrifugally cast in sizes of 30–48 in., inclusive. Such pipes are manufactured only to AWWA outside diameter B, in 16-ft laying lengths, for working pressures of 50, 100, 150, 200, and 250 psi, and classed according to the pressure rating. Although this was a manufacturer's standard, it was widely accepted and is in current use. Fittings are furnished either AWWA Class B or Class D all bell, or to either class in manufacturer's standard short-body fittings. Fittings with Class D metal and Class B socket and spigot diameter are sometimes used.

A considerable tonnage of cast-iron pressure pipe in 4–24-in. sizes is still produced according to Federal Specifications WW-P-421, although this standard is obsolescent in view of the newer ASA specifications and Federal Specifications WW-P-421a (*4*). When customers desire pipe of the same thickness as provided in Federal Specifications WW-P-421, the easiest transition is to specify the nearest standard class thickness of the ASA specifications and the proper diameter for use with a certain class of fittings.

During World War II, the Emergency Alternate Federal Specifications E-WW-P-421, dated Dec. 21, 1942, were published and used in the interest of conservation of strategic materials. All pipe wall thicknesses shown in these specifications were ASA standard class thicknesses, suitable for 150- and 250-psi working pressure, with 18/40 metal, trench condition B, and 5-ft maximum cover.

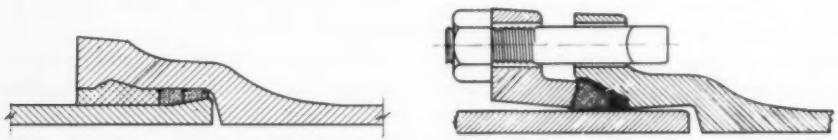

Fig. 1. Bell-and-Spigot and Mechanical Joints

Section at left shows bell-and-spigot joint, used as early as 1785. Mechanical joint, shown at right, is most widely used in current practice.

Of considerable and timely interest is the acceptance and promulgation by the General Services Administration of ASA designed pipe.

Federal Specifications WW-P-421a, dated Mar. 2, 1955, cover pipe designed in accordance with the ASA standard method of design in ASA A21.1. Tables are published for working pressures of 50–350 psi, using 18,000-psi minimum tensile in bursting, and 40,000-psi minimum modulus of rupture, 5-ft maximum cover, and laying condition B, in accordance with thicknesses shown in pit-cast pipe standard ASA A21.2, metal mold centrifugal pipe standard ASA A21.6 (AWWA C106), and sand-lined mold

centrifugally cast pipe standard ASA A21.8 (AWWA C108) (*5, 6*). For other conditions of cover and laying, and higher iron strengths, designs may be calculated from the standard practice manual ASA A21.1.

The bell-and-spigot, roll-on rubber ring, and standardized mechanical joints are all now included as pipe joint types I, II, and III, respectively.

The bell-and-spigot joint, which dates from 1785, and the mechanical joint, which was developed in 1927, are well known and widely used. Both types are illustrated in Fig. 1.

The mechanical joint was standardized by the cast-iron pipe industry in 1946, using the design of the "Double-X" joint developed by American Cast Iron Pipe Company, and is the joint most widely used at present for both water and gas.

The roll-on joint, shown in Fig. 2, is now a standard pipe industry joint in pipe sizes of 3–24 in., although it is also regularly produced in sizes of 30–48 in. It provides ease of assembly, joint-tightness, constant gasket compression, flexibility, and low cost. It has an excellent service record and is being used increasingly where a low-cost joint is desired and specifications exclude joints sealed with jute or paper in contact with water. This joint is also a development of American Cast Iron Pipe Company.

Data are presented on standard and "half-thickness" cement linings.* Thick linings are identical to those of ASA A21.4 (AWWA C104) (*7*) and half-thickness linings are, as the name implies, one-half as thick. The latter linings, with bituminous-seal coatings, have been used satisfactorily for over 23 years.

Half-thicknesses correspond to the various manufacturers' standards which have been marketed under a variety of trade names. Thickness for each pipe size are as follows: for pipe of 3–12-in. diameter, the thick lining is $\frac{1}{8}$-in. minimum, the half-thickness lining $\frac{1}{16}$ in.; for 14–24-in. pipe, $\frac{3}{16}$ and $\frac{3}{32}$ in.; and for 30–48-in. pipe, $\frac{1}{4}$ and $\frac{1}{8}$ in.

* Half-thickness cement linings are made under manufacturers' specifications only, and are not part of the ASA standards—*Ed.*

Development of ASA Specifications

In all of the foregoing formulas the tensile strength values are *assumed* strength properties of the pipe. The arbitrary additions were attempts to provide for foundry tolerances, corrosion, earth loads, hazards in handling and laying, and indeterminate stresses. In general, the wall thicknesses determined have been satisfactory. In some instances of deep cover or high internal pressure and poor laying conditions, the thicknesses are insufficient, whereas in instances of low and medium heights of cover, or low pressure and tamped backfill, they are excessive. In most instances, by actual experiment, the thicknesses are either too little, resulting in an inadequate design, or too great, causing an uneconomical design.

The old methods lacked a sound system of rating the strength of pipe. Three separate tests have been used to determine the tensile strength of iron in a pipe. A machined test bar, for example, from a separately cast specimen, or from the pipe wall itself, may

give a value of 40,000 psi. A rectangular test strip cut from the pipe wall, with two machined sides and two cast surfaces, when pulled in the tension test, may give a value of 35,000 psi. The most revealing test is the bursting test of full-length pipe. This test is not necessarily indicative of the true tensile strength of the iron in the pipe, but it provides a more nearly true value of the unit strength of the pipe, and, in comparison with the other two tests mentioned, may give a result of 18,000–25,000 psi for centrifugally cast pipe. These values are approximately one-half the strength of the Talbot strip cut from the pipe wall. Early formulas failed to recognize this.

Talbot Pipe Tests, 1926

As early as 1926 a research paper written by Arthur N. Talbot and Frank E. Richart brought out the relations between the properties of pipe under tests and the corresponding properties found in test specimens from the same pipe tested (*8*). Another paper by Talbot showed that the internal-pressure bursting test is one of the best actual indicators of quality and revealed that pipes manufactured by different methods gave different strength results (*9*).

Sectional Committee A21

Research work was furthered by the formation of ASA Sectional Committee A21 in 1926. The sponsor bodies were American Gas Association, ASTM, AWWA, and NEWWA. The committee was composed of representatives of the sponsor bodies, interested pipe manufacturers, and members of other national associations.

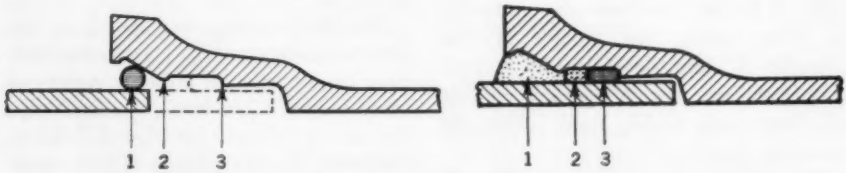

Fig. 2. Roll-On Joint

Section at left shows position of pipe and gasket before assembly with (1) round rubber gasket, (2) restriction ring cast in bell, and (3) positive gasket seat. In section of assembled joint at right, (1) asphaltic sealing compound, (2) strand of braided jute behind restriction ring, and (3) rubber gasket compressed to half diameters are shown.

New Design Methods

Intensive research performed at the University of Illinois and Iowa State College finally evolved a law of pipe design based on many full-scale tests of pipe under actual internal-pressure and earth load service conditions. The testing procedures and derivation of the actual law of design are too involved to be presented here but can be studied in detail in publications relating to the law (*10*).

The method of design, ASA specifications A21.1, represents the method

considered most satisfactory by the authorities in this field. The design method is distinct from the older standards and corrects their faults by embodying (in addition to the provisions for internal pressure and water hammer) the following:

1. Consideration of the earth load tending to produce crushing action on the pipe, thus reducing the pipe's bursting strength
2. Consideration of the effect of laying or bedding condition of the pipe in reducing or increasing the effect of the earth load
3. Consideration of the effect of surface loads, such as those due to passing vehicles, which may be added to the earth load
4. The rating of the ability of the various types of pipe to resist internal pressure and external crushing loads (acting singly or simultaneously)
5. Suitable factors of safety and definite allowances for foundry tolerance and corrosion.

These specifications gave the engineer, for the first time, the normal tools he uses in design—namely, a compilation of material strengths, a proved method of determining and applying loads, and reliable laboratory checks to insure conformance to specifications and reveal proof of quality.

Following the natural progression from older methods of pipe production to modern methods, utilizing irons of greater strength, it is to be expected that, in general, wall thicknesses would be reduced somewhat. In following the natural laws of scientific design, however, the wall thicknesses vary considerably as befits the loading demands. Where internal pressure and external loads are moderate, pipe with somewhat decreased wall thickness will do the job reliably. On the other hand, for higher pressures and heavier external loads, much thicker pipe is shown necessary in conditions where thinner pipe (produced to old Federal Specifications WW-P-421 for centrifugally cast pipe) had formerly been used without distinction.

The transition from AWWA specification pit-cast pipe to Federal Specifications WW-P-421 centrifugally cast pipe of greater strengths resulted in reduced pipe wall thicknesses which have proved more than ample in service and are used without hesitation under widely varying pressures and field conditions.

As an example, 12-in. AWWA Class B pit-cast pipe having a wall thickness of 0.62-in. was rated at 86-psi working pressure without limitation as to trench load. Federal Specifications WW-P-421 centrifugally cast 12-in. pipe with a wall thickness of 0.50 in. is rated at 150-psi working pressure without limitation as to trench load. Centrifugally cast 12-in. pipe made to ASA specifications with 18/40 metal strength for 150-psi operating pressure plus 110-psi water hammer, 5 ft of cover, and laid in a flat-bottom trench with tamped backfill (laying condition B), is 0.48-in. thick.

The ASA specification for similar service conditions (except 12 ft of cover) requires 0.56-in. wall thickness. It is obvious that the ASA law of design follows a more logical pattern than those of the older specifications, yet is more conservative regarding earth loads.

Actually, 18/40 pipe designed according to ASA A21.1 for 150-psi operating pressure, 5 ft of cover, and laying condition B, closely parallels

the thickness of pipe designed to Federal Specifications WW-P-421 in 4–12-in. sizes. In larger sizes, wall thicknesses become somewhat less for the same pressure ratings when the cover factor is constant.

Another consideration of importance is that of larger inside diameters of ASA pipe utilizing irons of greater strength. As the AWWA outside diameter is maintained, any reduction in wall thickness increases the inside diameter beyond the nominal size, causing an appreciable increase in area and hence capacity. The factor of greater capacity should be weighed when considering pipe of other materials, such as steel, in which the nominal size is often the actual outside diameter, or nonmetallic alternates in which the inside diameter may be only equal to, or even less than, the nominal size.

Methods for Computation of Strength and Thickness

A series of comprehensive experiments at the University of Illinois and Iowa State College developed the relations of combined tensile and flexural stress to ultimate tensile and bending strength of cast-iron pipe. The ultimate values of several different kinds of pipe were first established by determining their bursting and crushing strengths through a sufficient number of tests to determine average values for each group of pipe. Subsequently, tests to destruction were made on similar lots of pipe in which internal pressure and external load were acting simultaneously. In each lot of tested pipe the values of the stresses to produce failure under combined internal pressure and external load showed a definite relation to the ultimate values of these stresses acting singly.

Tensile Strength

The ultimate tensile strength used in designing a pipe wall thickness was decided upon as the value established by the *bursting tests of full-length pipe*. Compared to previous formulas, in which tensile strengths were assumed, or indicated by separately cast test bars, this was indeed a forward step. The minimum tensile strength values, derived from actual tests, were, for pit-cast pipe, 11,000 psi; and for centrifugally cast pipe, 18,000 psi.

Modulus of Rupture

The strength of pipe to resist crushing is rated by a standard compression test known as the "three-edge bearing test." The modulus of rupture for pipe designed by the ASA method was chosen as that developed in the standard ASTM three-edge bearing test as illustrated in Fig. 3. A sufficient number of tests were made to establish average values. The minimum values then established were, for pit-cast pipe, 31,000 psi, and for centrifugally cast pipe, 40,000 psi.

The manufacturer must periodically run such bursting and ring tests in conjunction with strip tests so that he can certify the design values of the modulus of rupture and the tensile strength of the iron in the pipe.

Modulus of Elasticity

The acceptance values for the secant modulus of elasticity are determined by tests on Talbot strips cut from 3–24-in. pipe, and are as follows:

1. Pit-cast pipe (ASA A21.2) : 10,000,000 psi maximum
2. Pipe centrifugally cast in metal molds (ASA A21.6) : 12,000,000 psi maximum
3. Pipe centrifugally cast in sand-lined molds (ASA A21.8) : 10,000,000 psi maximum.

In each category, whenever the modulus of elasticity exceeds the stated maximums, the modulus of rupture must exceed the minimum requirements for each specification by at least the same percentage.

The same specifications apply to pit-cast pipe in sizes of 30-in. and larger. The acceptance tests for centrifugally cast pipe in 30–48-in. sizes are to be made on the ASTM standard 2-in. diameter test bars which must have a minimum breaking load of 6,000 lb, and a deflection of 0.15 in. plus 0.01 in. for each 500 lb of breaking load in excess of 6,000 lb. Use of Talbot strips is optional.

Safety Factor

A factor of safety of 2½ has been agreed upon for both external loads and internal pressure, *acting simultaneously*. This includes the allowance for cover burden, truck load, or water hammer, as the design may require. These safety factors apply in calculating the *net* wall thickness required to resist all forces acting simultaneously on the pipe. To the net thickness is added the proper corrosion allowance and manufacturing tolerance.

Laying Conditions

The six chief field conditions of laying pipe are shown in Fig. 4. Conditions E and F are uncommon in cast-iron pipelines. The most popular of the other four is condition B, which is considered the best for developing the strength of the pipe and at the same time maintaining an economical cost of trenching and laying.

Because the methods of supporting the pipe in the field under different laying conditions are not the same as used in the laboratory test, it was necessary to develop a correlation between the two. Tests were made, therefore, at Iowa State College to determine the "ratios to three-edge bearing" for each of the field conditions of laying. These ratios establish a relationship between

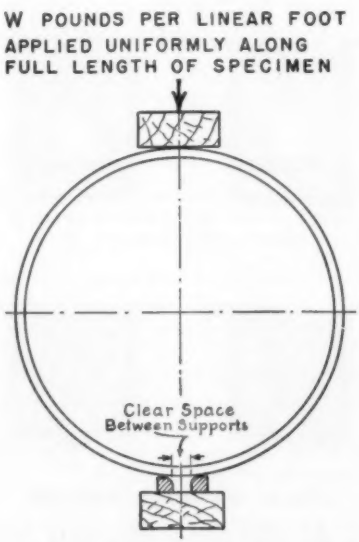

Fig. 3. Three-Edge Bearing Load

The condition of loading shown above is referred to in ASA A21.1 (in which the illustration appears as Fig. 2) as the "three-edge bearing load" because two supports close together at the bottom are used to facilitate laboratory tests by preventing the specimen from rolling.

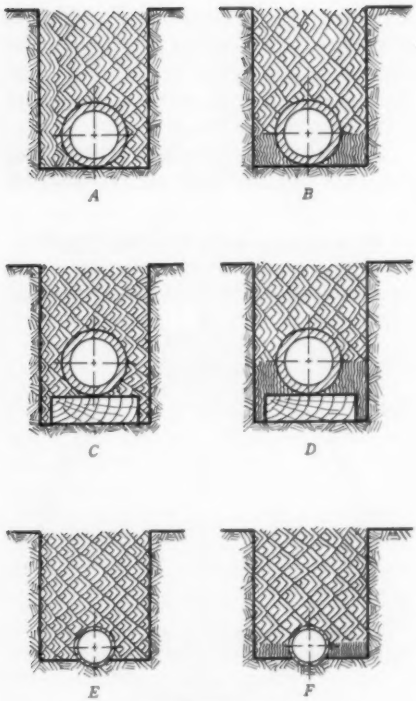

Fig. 4. Pipelaying Conditions

The six chief field conditions of laying pipe, as shown above, are: A—*flat-bottom trench with untamped backfill;* B—*flat-bottom trench with tamped backfill;* C—*pipe laid on blocks with untamped backfill;* D—*pipe laid on blocks with tamped backfill;* E—*shaped-bottom trench with untamped backfill;* F—*shaped-bottom trench with tamped backfill.*

actual field laying conditions and the standard laboratory test. The trench load divided by the ratio gives a revised load that will have the same effect on a pipe subjected to three-edge bearing as the actual trench load does on a pipe for a given field condition.

The ratio is greater than unity for pipe laid in a flat-bottom trench with either tamped or untamped backfill, so the loads are reduced. For pipe supported on blocks, with untamped backfill, the ratio is less than unity, increasing the load and showing the destructive effects of this type of laying.

Load-Pressure Relationship

As established in the tests, a pipe failure may be caused by internal pressure alone, by trench load alone, or by any number of combinations of internal pressure and external loads acting together. If the internal pressure is high, the trench load required to produce pipe failure is low, and conversely. This relationship has been represented as a parabolic load-pressure curve, shown in Fig. 5. In this graph, P is the ultimate bursting pressure that will cause failure of the pipe when the external load is zero. Similarly, the ultimate external load, W, is that which will cause the pipe to fail when the pressure is zero. The term w is the external load which will cause failure when the internal pressure is p for a given pipe diameter and *net* wall thickness. The parabolic curve indicates the percentage of these ultimate forces which will cause failure under an infinite number of combinations of internal pressure and external loads acting simultaneously.

The solid-line rectangle indicates one combination of internal pressure and external load which, acting simultaneously, will cause failure of the pipe. The broken-line rectangle indicates the percentage of ultimate strength values to which the pipe wall metal is subjected when a factor of safety of $2\frac{1}{2}$ is used, *based on the combination of these two forces acting simultaneously.* Actually, the pipe wall metal is stressed to only 20 per

cent of its ultimate bursting strength and 29 per cent of its ultimate external-load strength. The factor of safety for bursting alone is therefore 5, and this factor is based on the net design thickness before any additions for manufacturing tolerance and corrosion allowance. The factor of safety would obviously be increased if based on the nominal wall thickness. Factors of safety for the normal operating pressure of city water systems, with cover up to 5 ft for pipe of nominal wall thickness, will be about 12 or more for sizes of 12-in. and smaller, and 7 or more for pipe of larger diameters, assuming of course no deviation from standard class thickness for tolerance or corrosion.

The factor of safety, which applies to both internal pressure and external load, should not be confused with safety factors applicable to the thin-tube formula or thick nonmetallic conduits considered only from the standpoint of internal pressure and ultimate tensile strength.

Earth Loads

Anson Marston, the late dean of Iowa State College, began many years ago a series of full-scale tests of earth loads on pipe of various kinds. The work has been continued by his successors and reported in many Iowa State College bulletins. ASA Sectional Committee A21 took advantage of this experience and technique by employing W. J. Schlick, who was in charge of the work, to make the extensive tests which form the basis of the committee's method of design.

The earth load can be determined by a basic formula but is much more easily derived from the basic earth load graph shown in Fig. 6. The fill material is assumed to weigh 120 lb per cubic foot. In relatively shallow trenches the column of earth directly above the pipe weighs on the pipe vertically. This weight is increased by the internal friction between the fill material settling around the sides of the pipe and the fill material directly over the pipe. There is a critical height of fill for each pipe size and settlement ratio, where the internal friction of the fill material exactly neutralizes the tendency of side fill to settle more than the column of earth directly above the pipe. The point of

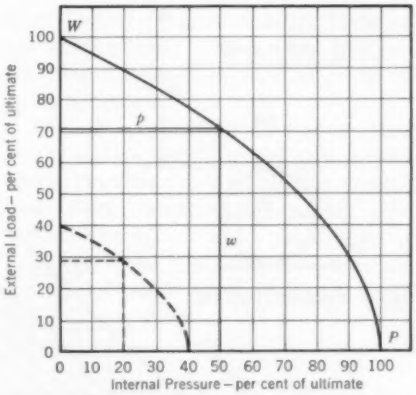

Fig. 5. Application of External Load, Internal Pressure, and Safety Factor to Cast-Iron Pipe Design

Parabolic curve for the formula:

$$w = W\sqrt{\frac{P - p}{p}}$$

is shown, in which P *is ultimate bursting pressure that will cause pipe failure when the external load is zero;* W *is the ultimate external load, or that which will cause pipe failure when the pressure is zero; and* w *is the external load that will cause failure at internal pressure* p *for a given pipe wall diameter and net wall thickness.*

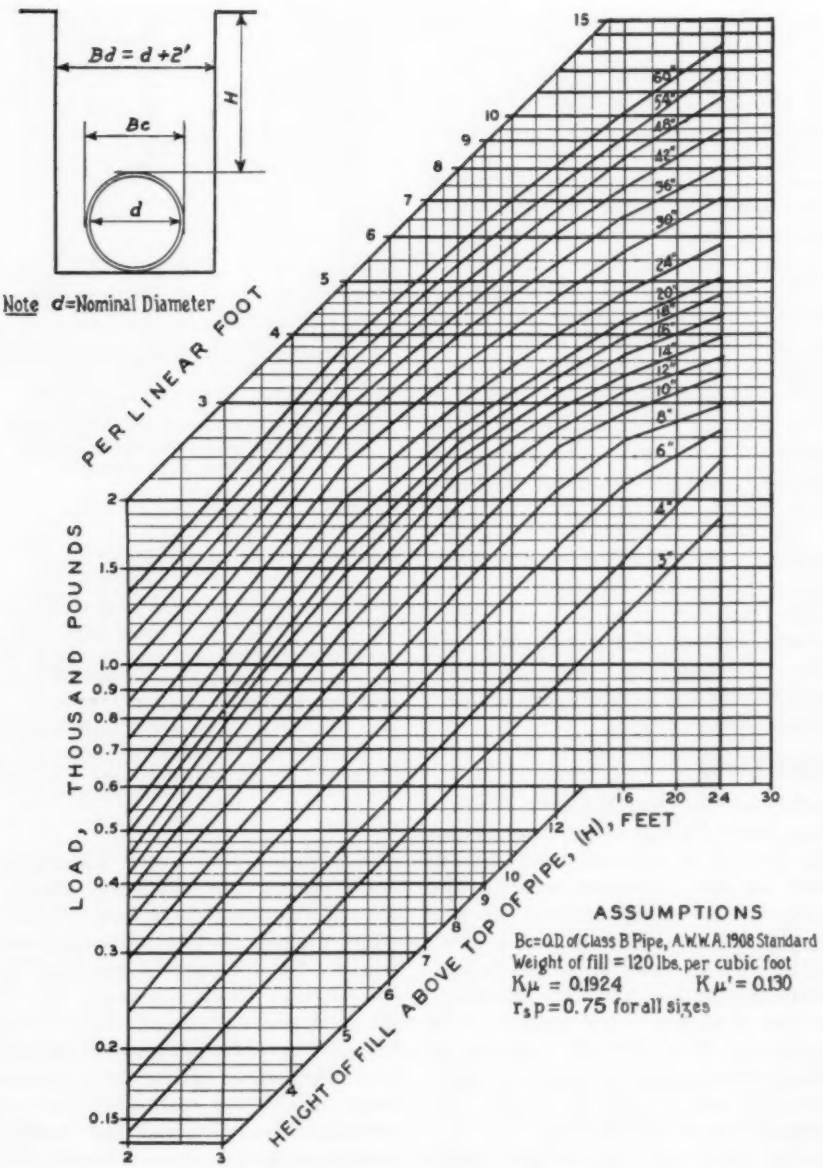
Bd = d +2'
Bc
d
H
Note d=Nominal Diameter
LOAD, THOUSAND POUNDS PER LINEAR FOOT
HEIGHT OF FILL ABOVE TOP OF PIPE, (H), FEET
ASSUMPTIONS
Bc=O.D. of Class B Pipe, A.W.W.A. 1908 Standard
Weight of fill = 120 lbs. per cubic foot
Kμ = 0.1924 Kμ' = 0.130
r_sp = 0.75 for all sizes

Fig. 6. Basic Earth Load Graph

Diagram shown is Fig. 4 of ASA A21.1 (p. 9), which is used to determine earth loads on pipe for width of ditch d + 2 *ft, field conditions A, B, C, E, and F.*

this bridging effect is called the "height of equal settlement." The load on the pipe caused by the fill above this point is not increased by the internal friction of the fill material. If the height of fill is much greater than the height of equal settlement, the load on the pipe is lessened by the effect of the friction of the fill material against the sides of the ditch. The depth-of-cover calculations are quite conservative, compared to the older specifications.

In compiling the standard pipe thickness tables, ASA Sectional Committee A21 determined earth loads by using a trench width equal to the nominal diameter of pipe (in feet) plus 2 ft, expressed as $d + 2$ ft. Field experience has shown that many contractors prefer to cut trenches much narrower than $d + 2$ ft. It is not uncommon for trenches to be $d + 1$ ft, which lessens the trench loading, resulting in reduced pipe wall thickness. The committee recognized this fact by including in specifications A21.1 a diagram for obtaining earth loads on pipe for trench widths of $d + 1$ ft. This diagram may be used in calculating pipe thicknesses.

Truck Load and Impact

In addition to consideration of backfill load, the external load includes allowance for truck traffic based on two trucks passing each other with adjacent rear wheels 3 ft apart and a load of 9,000 lb on each rear wheel, plus 50 per cent impact. This load is multiplied by a surface load factor for each pipe size which depends upon the depth of backfill to the top of the pipe. The resulting load per foot is then modified, depending on laying condition. Generally, truck load dominates for 5 ft or less of cover and water hammer for deeper cover. The truck load on the pipe decreases rapidly as depth of cover increases.

Because the chance is remote that water hammer and truck load would occur simultaneously, only the condition which demands the greater pipe wall thickness is used. In design of gas pipe, truck load allowance is made for all calculations, as obviously there is no need to consider any hammer effect.

Pipe Wall Thickness

The strength of pipe in crushing when tested by a center load in three-edge bearing (*see* Fig. 3) is expressed as modulus of rupture, R, and is calculated by the formula:

$$R = \frac{0.0795(d + t)W}{t^2}$$

in which d is the ID of the pipe, in inches; t is the average thickness, in inches; and W is the breaking load, in pounds.

When a sufficient number of tests have been made on pipe of a certain type of manufacture, in a range of sizes and thicknesses, a definite value of R can be set for that product. The crushing strength of any size or thickness of pipe of this type can then be determined by the transposed formula:

$$W = \frac{Rt^2}{0.0795(d + t)}$$

Similarly, the bursting tensile strength of pipe of a certain type is obtained by numerous tests, as previously explained. When the minimum tensile strength value of the type of pipe is determined, then the bursting

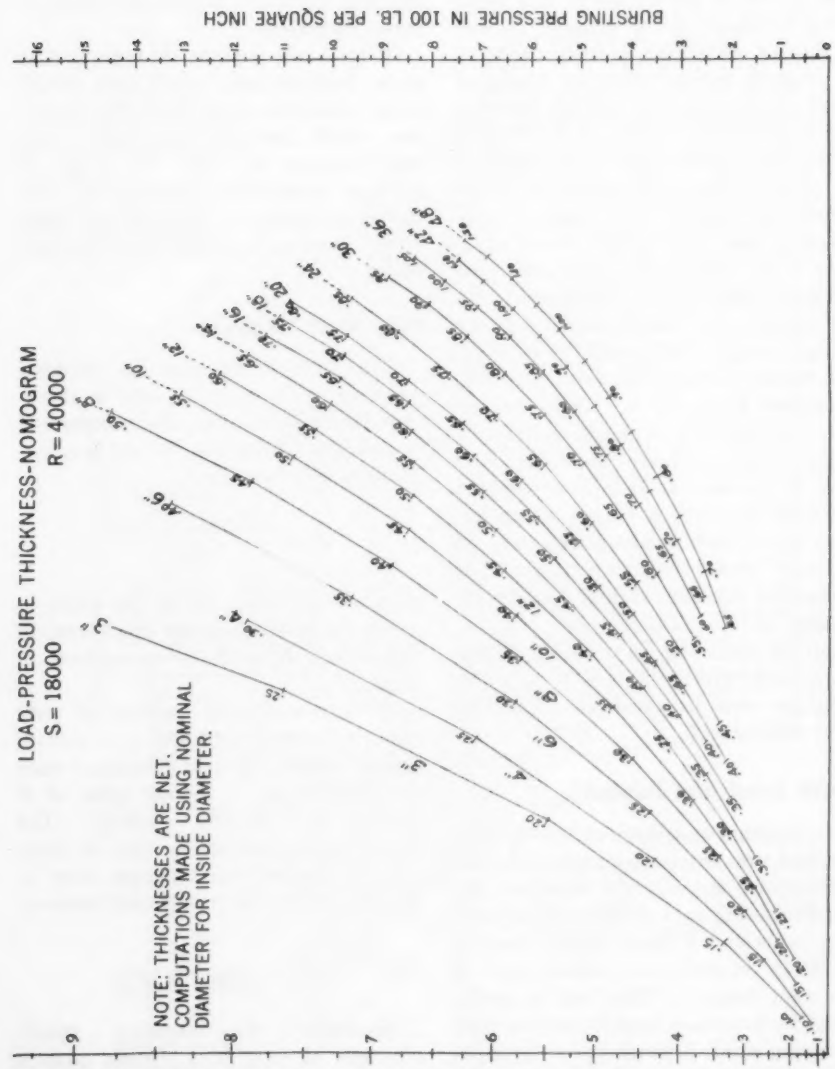
BURSTING PRESSURE IN 100 LB. PER SQUARE INCH
LOAD-PRESSURE THICKNESS-NOMOGRAM
S = 18000
R = 40000
NOTE: THICKNESSES ARE NET.
COMPUTATIONS MADE USING NOMINAL
DIAMETER FOR INSIDE DIAMETER.
CRUSHING LOAD IN THOUSAND LB. PER LINEAR FOOT UNDER 3-EDGE BEARING CONDITIONS

Fig. 7. Nomogram for Computation of Pipe Wall Thicknesses

pressures may be calculated for all classes and sizes of pipe of this type by the formula:

$$P = \frac{2St}{d}$$

in which P is the bursting pressure in pounds per square inch, and S is the minimum tensile strength value of the general type.

The equations used in determining pipe thicknesses are too involved for direct mathematical solution. Using the above formulas, however, and substituting in the combined load-pressure equation, a solution can be obtained by trial and error.

The load-pressure equation is:

$$w = W\sqrt{\frac{P - p}{P}}$$

in which W is the ultimate crushing load, converted to equivalent 3-edge bearing value, in pounds per linear foot; P is the ultimate bursting pressure, in pounds per square inch; and w and p are the crushing load and internal pressure, respectively, which, acting together, are just sufficient to cause failure.

In using the above equation, a reasonable pipe thickness is assumed and, after other values are calculated, the right-hand term must just be equal to or slightly larger than the left-hand term for a correct solution to obtain net thickness. To the net (or design) thickness must be added corrosion allowance and foundry tolerance.

Other methods for determining pipe thicknesses include plotting curves and estimating thicknesses by interpolation, and use of the squared scale. The easiest and most rapid of all methods is the use of the nomographic system developed by C. K. Donoho, chief metallurgist of American Cast Iron Pipe Company. In this method, a straightedge connecting total external load to total internal pressure ordinates passes through approximate parallel lines representing thickness values which can be easily picked off, as shown in Fig. 7.

Corrosion and Foundry Tolerance

The second term of the Brackett, Fanning, and Fairchild formulas was a rule-of-thumb attempt to provide for handling, foundry tolerance, corrosion or any similar indeterminate factor. It had no scientific basis.

TABLE 2

Maximum Foundry Tolerances

Nominal Diam. of Pipe *in.*	Pit Cast	Centrifugal Process
	Tolerance—*in.*	
3–8	0.07	0.05
10–12	0.08	0.06
14–24	0.08	0.08
30–60	0.10	0.10*

* For 30–48 in. only.

Committee A21 made an extensive study of the corrosion of cast-iron pipe in long-time service and, as a result, a corrosion allowance of 0.08 in. was decided upon for the standard calculation. This allowance, which is generous (except under unusually corrosive conditions), is added to the net thickness. If corrosive conditions are well known, the allowance may be greater or less for certain environments.

TABLE 3

*Standard Pipe Wall Thickness Classes**

Nominal Diameter of Pipe *in.*	Pipe Wall Thickness (*in.*) for Standard Thickness Class No:										
	20	21	22	23	24	25	26	27	28	29	30
3	—	—	*0.32*	0.35	0.38	0.41	0.44	0.48	0.52	0.56	0.60
4	—	0.32	*0.35*	0.38	0.41	0.44	0.48	0.52	0.56	0.60	0.65
6	0.32	0.35	*0.38*	0.41	0.44	0.48	0.52	0.56	0.60	0.65	0.70
8	0.35	0.38	*0.41*	0.44	0.48	0.52	0.56	0.60	0.65	0.70	0.76
10	0.38	0.41	*0.44*	0.48	0.52	0.56	0.60	0.65	0.70	0.76	0.82
12	0.41	0.44	*0.48*	0.52	0.56	0.60	0.65	0.70	0.76	0.82	0.89
14	0.43	*0.48*	0.51	0.55	0.59	0.64	0.69	0.75	0.81	0.87	0.94
16	0.46	*0.50*	0.54	0.58	0.63	0.68	0.73	0.79	0.85	0.92	0.99
18	0.50	*0.54*	0.58	0.63	0.68	0.73	0.79	0.85	0.92	0.99	1.07
20	0.53	*0.57*	0.62	0.67	0.72	0.78	0.84	0.91	0.98	1.06	1.14
24	0.58	*0.63*	0.68	0.73	0.79	0.85	0.92	0.99	1.07	1.16	1.25
30	0.68	*0.73*	0.79	0.85	0.92	0.99	1.07	1.16	1.25	1.35	1.46
36	0.75	*0.81*	0.87	0.94	1.02	1.10	1.19	1.29	1.39	1.50	1.62
42	0.83	*0.90*	0.97	1.05	1.13	1.22	1.32	1.43	1.54	1.66	1.79
48	0.91	*0.98*	1.06	1.14	1.23	1.33	1.44	1.56	1.68	1.81	1.95

* Italicized numerals indicate minimum thicknesses for 18/40 pipe in accordance with ASA specifications A21.6, A21.7, A21.8, and A21.9. Other thicknesses shown for classes 20 and 21 can be furnished in iron pipe of higher tensile strength.

Foundry tolerance for pipe thicknesses depends somewhat upon the method of manufacture. Recommended tolerances are shown in Table 2. As these allowances vary according to the manufacturing process used and the control exercised, the allowances may be either greater or less than the suggested figures.

Some engineers specify minimum calculated thicknesses including corrosion allowance but not tolerance, and permit foundries to cast as close to the minimums (but no less) as their particular practice will permit.

The net design thickness plus corrosion allowance plus tolerance is the calculated thickness which can be used for manufacture or adjusted to the nearest standard-class thickness.

Standard Thickness Classes

Minimum thicknesses have been determined on the basis of economical manufacture and handling hazards. The calculated thicknesses in small-diameter pipe may be appreciably less than the minimum standards, in which the minimum class thickness governs. After the calculated thickness has been established, reference must be made to the standard class thickness to select that nearest the calculated thickness.

Standard class numbers have been established for wall thicknesses in all pipe sizes. Those for cast-iron pipe centrifugally cast are shown in Table 3. The thicknesses of successive classes are 8 per cent greater in each case than those of the next lower class. These classes were established to prevent the necessity of stocking pipe of too many different wall thicknesses.

Calculated thicknesses are adjusted to the nearest class thickness. These class thicknesses should be specified in ordinary practice. In long lines, how-

ever, or when pipe of large diameters is used, it may be more economical to employ the calculated thickness rather than standard class thicknesses. In the majority of installations, published thicknesses for a variety of load and pressure conditions will suffice. For unusual conditions, however, the law of design can be used to calculate thicknesses required. Figure 8 shows a comparison of the new ASA specifications and the older standards.

Current Specifications

Crowning the achievements of ASA Sectional Committee A21 was the acceptance of the A21 series of specifications by the four sponsoring associations in 1952 and 1953. The specifications now available are listed in Table 4. As is seen, an easily remembered numerical cross reference was evolved. Methods of manufacture, strength values, and specification numbers are shown.

As has been previously noted, the revised AWWA specifications of 1908 are now AWWA C100 and cover only bell-and-spigot fittings.

Bell-and-Spigot Pipe

The new ASA specifications for bell-and-spigot water pipe provide for pipe to be made so that it will readily connect with any pipe previously furnished to older AWWA or federal specifications. ASA outside diameters are related to AWWA specification sizes in Table 5.

Pipe of 4–12-in. sizes can be produced to Class D or alternative diameters, resulting in a gain in hydraulic capacity and only a slight reduction in fittings joint space. These diameters, however, are rarely used.

All pipes of 4–12-in. size, regardless of outside diameter, are used with AWWA C100, Class D, or C110, Class 250, short-body fittings. For 14–24-in. sizes, two classes of AWWA fittings are available—B and D. Normally the Class B fittings rated at 86 psi are used (with the exception of some wyes and full-opening tees and crosses) with up to 100-psi working pressure for pipe made on the B pattern. Above this pressure, AWWA Class D fittings (rated at 173 psi at present) are used for pipe made from the larger diameters, or D patterns.

For pipe of 30 in. and larger, centrifugally cast and made to B diameter, either Class B bell-and-spigot or Class D all-bell fittings can be used, although the joint space on the D fittings is quite thick. Fittings of both classes can be made to AWWA C100 or manufacturer's standard short-body fittings. Very often Class "B/D" fittings are used—this designation signifying a fitting of D metal thickness with Class B socket and spigot diameters.

Bell-and-spigot pit-cast pipe for water in sizes of 30–60 in. is produced on AWWA patterns A, B, C, D, E, and F with outside diameters determined by the wall thickness of the pipe in accordance with ASA specifications A21.2.

Class 150 bell-and-spigot pipe in sizes of 14–24 in., produced to B diameter for use with Class B fittings, is very popular in filtration and sewage treatment plants where pressures are relatively low. As fittings are the most expensive part of a line, the use

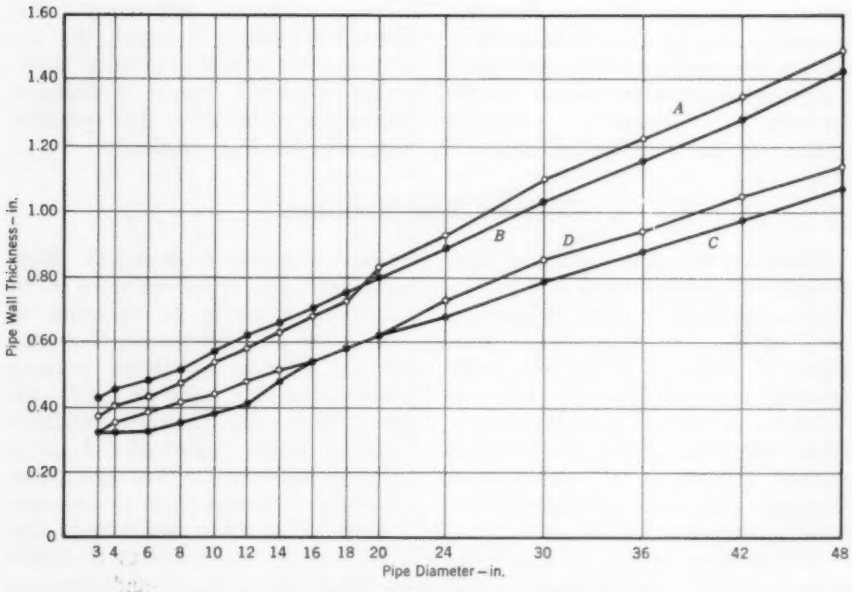

Fig. 8. Pipe Wall Thickness Comparison Graph

Pipe wall thicknesses for 150-psi working pressure plus water hammer, 5-ft cover, laying condition B, are shown for the following specification designs:

A—*ASA A21.2 pit cast, 11/31 metal strength*
B—*AWWA 1908 Class B, pit cast*
C—*ASA design centrifugally cast, 21/45 metal strength*
D—*ASA A21.6 and A21.8 centrifugally cast, 18/40 metal strength.*

of Class B fittings, in such cases, reduces the weight considerably and is therefore more economical.

With long supply lines involving comparatively few fittings, the higher cost of pipe of a larger diameter must be balanced economically against the increased capacity of the line.

As can be seen, pressure ratings of pipe far exceed those of fittings; however, the above usage is common practice. The AWWA specifications C100 are an interim recommendation, as Sectional Committee A21 has been unable to complete the specifications on fittings, except for 3–12-in. short-body type for 250-psi maximum working pressure, plus water hammer. It is hoped the specifications for fittings, based on further tests, will be forthcoming in a year or two and will resolve questions of matching fittings with pipe, as to both strength and joint space. High-strength nodular iron may offer some solution to these problems, as well as strengthening wyes and certain tees and crosses.

Mechanical and Roll-On Joint Pipe

Mechanical-joint pipe, although originally designed for gas, is now the most widely used pipe for water and other liquid service. For water service, the mechanical joint must be specified when referring to a water pipe standard. For centrifugal pipe, the

best procedure is to specify ASA A21.6 or ASA A21.8, with standardized mechanical joint (ASA A21.11) or Federal Specifications WW-P-421a, Type II joint. The A21.11 specifications cover the joint only, not the pipe barrel. The dimensions of standardized mechanical-joint bells are shown in ASA A21.11.

Standardized mechanical-joint and roll-on joint pipe are produced to only one diameter for each size. These diameters correspond to the ASA standard diameters in sizes of 3–12 in. and 30–48 in., and to the alternate ASA diameters in sizes of 14–24 in.

There are some slight variations in wall thicknesses of gas pipe as compared to water pipe. Therefore, when centrifugally cast gas pipe is required, ASA specifications A21.7 or A21.9 should be used with standardized mechanical joint (ASA A21.11).

Mechanical-joint pit-cast pipe is produced on AWWA pattern B in sizes of 30–48 in. inclusive, using ASA A21.2 thicknesses. At the present time 48 in. is the largest diameter of mechanical-joint pipe available.

TABLE 4

Current Specifications for Cast-Iron Pipe

Specifications		Subject	Lengths *ft*	Min. Design Values *1,000 psi*	
ASA	AWWA			Bursting Tensile	Modulus of Rupture
A21.1	C101	Computation of strength and thickness of cast-iron pipe	—	—	—
A21.2*	C102	Cast-iron pipe, pit-cast, for water or other liquids	12	11 18§	31 40§
A21.4	C104	Cement-mortar lining for cast-iron pipe and fittings	—	—	—
A21.6†	C106	Cast-iron pipe, centrifugally cast, in metal molds, for water or other liquids	12 and 18	18	40
A21.8‡	C108	Cast-iron pipe, centrifugally cast in sand-lined molds, for water or other liquids	16, 16½ and 20	18 21§	40 45§
A21.10	C110	Short-body cast-iron fittings, 3–12 in., for 250-psi water pressure plus water hammer	—	—	—
A21.11	C111	Mechanical joint for cast-iron pressure pipe and fittings	—	—	—

* Corresponding specifications for gas are ASA A21.3.
† Corresponding specifications for gas are ASA A21.7.
‡ Corresponding specifications for gas are ASA A21.9.
§ Strength values exceed ASA minimum, but are those most used to design pipe in accordance with ASA A21.1. This is also true of pipe designed according to corresponding gas specifications A21.9.

AWWA outside diameters and short-body fitting classes are shown for both mechanical-joint and roll-on joint pipe in Table 6.

It is necessary that any connection between bell-and-spigot pipe and mechanical-joint pipe be carefully checked to insure matching. There is only one class of fittings for each size of mechanical-joint pipe, although some manufacturers will furnish heavier-wall fittings where required.

Use of ASA Tables

In using the ASA pipe tables, the following procedure is advised:

1. Determine internal operating pressure, laying condition, strength, depth of cover, and whether the truck load or water hammer factor is to be used.
2. Select the proper thickness from the table for the combination of working conditions.
3. By determining what class of fitting (in bell and spigot) is to be used, select pipe of the correct pattern to socket the fittings.
4. Refer to class number for thickness required and specify pipe according to class and outside diameter.

TABLE 5

Outside Diameters for Bell-and-Spigot Pipe ASA Specifications

Nominal Diam. of Pipe *in.*	OD (AWWA Classes)	Working Pressure *psi*
3	3.96 in.	all pressures
4, 6	Class A	all pressures
8, 10, 12	Class B	all pressures
14–24	Class B	50, 100
14–24	Class D	150–350
30–48	Class B	50–250

For those who prefer to continue using a certain thickness of pipe to some older specification, it is only necessary to determine and specify the nearest ASA standard class to the thickness required.

To specify bell-and-spigot pipe, determine the above and, as an example, specify as follows: 16 in. ASA A21.6 or A21.8, 18/40 metal, Class 22, AB pattern; or, for standardized mechanical-joint pipe: 16 in. ASA A21.7 or A21.9, 18/40 metal, Class 22.

For purposes of illustration, let it be supposed that 30-in. bell-and-spigot pipe is needed in a city water distribution system for 150-psi working pressure, 3 ft of cover, laying condition B, with allowance for water hammer. The design conditions should be detailed, and the various specifications under which the material could be furnished are as follows:

1. Pipe to be centrifugally cast with thickness designed in accordance with the modified Fairchild formula, as used in Federal Specifications WW-P-421.
2. Pipe to be centrifugally cast with thickness designed in accordance with ASA A21.8 with 18/40 metal, or higher (centrifugally cast pipe can be furnished in 18/40 or 21/45 metal).

TABLE 6

OD and Fitting Classes for Standardized Mechanical-Joint and Roll-On Joint Pipe (Centrifugally Cast or Pit Cast)

Nominal Diam. *in.*	OD (AWWA Classes)	Short-Body Fittings
3	3.96 in	Class 250 (ASA A21.10)
4–6	Class A	Class 250 (ASA A21.10)
8–12	Class B	Class 250 (ASA A21.10)
14–24	Class B	Class 150 Short Body*
30–48	Class B	Class 150 Short Body*

* This is an industry standard, and manufacturer should be consulted for pressure rating. Where desired, manufacturers will furnish heavier-bodied fittings for higher working pressures.

3. Pipe to be pit cast with wall thickness designed in accordance with ASA A21.2 with 11/31 metal, or higher (some pit-cast pipe manufacturers now offer both 11/31 and 18/40 pipe).

In the above examples, pipe used should be of the thicknesses shown in Table 7. As the table indicates, a variety of thicknesses may be used and the customer may be penalized unless he can familiarize himself with the specifications and know what to use.

The higher strength values result in a reduction in wall thicknesses and increased economy, or, when used in a specification for a heavy wall, in greater factors of safety.

A national survey indicated that pipe furnished for cover of up to 5 ft would be satisfactory for most normal pipe cover in almost all states. Detail tables have therefore been published for this cover, under various pressures, for laying conditions A, B, C, and D, and for the several strengths of iron available.

There will of necessity be a transition period from all old standards to the modern specifications. It is earnestly hoped by the manufacturers that this can be accomplished with a minimum of confusion. The detailed tables published by ASA, AWWA, and various manufacturers are quite easy to follow. Once established it is believed that the more recent specifications will ultimately offer a greater aid to engineers and designers and increased flexibility with which to meet the many varied operating conditions.

TABLE 7

Metal Strengths and Thicknesses Used in Specifying Cast-Iron Pipe

Length ft	Nominal Diam. in.	Class	Casting Method	Specifications	Strength of Iron	Metal Thickness in.
16	30	150	centrifugal	modified Fairchild formula	—	— 0.89
16	30	150	centrifugal	ASA A21.8	18/40	0.79
16	30	150	centrifugal	ASA A21.1	21/45*	0.73
12	30	150	pit cast	ASA A21.2	11/31	1.10
12	30	150	pit cast	ASA A21.1	18/40*	0.79

* ASA A21.8 shows only 18/40 metal and A21.2 only 11/31 metal. Metal of greater strengths may be used to design according to ASA A21.1.

Sample Calculation

A sample calculation is offered in order to familiarize the reader with the techniques of using the charts and tables of the newer specifications. The problem is to determine the wall thickness of a 24-in. pipe for the following conditions: 100-psi working pressure; 5-ft cover depth plus truck load; laying condition B; using 18/40 metal. Charts and tables should be used as noted.

External Loads

1. On the chart shown in Fig. 6, a tracing of the 24-in. curve to the coordinates representing 5-ft cover depth shows an earth load of 2,011 lb per linear foot.

2. Allowing 9,000 lb per wheel plus 4,500 lb per wheel for impact (50 per cent), and referring to the surface load chart shown in Fig. 9, which indicates a factor of 0.067 for 24-in. pipe with 5-ft cover, the surface load for two passing trucks is calculated as 905 lb per foot.

3. Referring to Table 8, it is found that, for the conditions specified, 85 per cent of the calculated truck load represents the actual truck load, or 769 lb per foot.

4. Adding the earth load (2,011 lb) to this figure, the result is 2,780 lb per foot.

5. Referring to Table 9, the factor for correction to three-edge bearing is found to be 1.52. Dividing 2,780 lb by this factor, the equivalent three-edge bearing load is 1,829 lb per linear foot.

6. Using the fixed safety factor of $2\frac{1}{2}$, the total external load, corrected for all specified conditions, is 4,572 lb per linear foot. (The total external load without allowance for truck load is 2,011 divided by 1.52 times $2\frac{1}{2}$, or 3,307 lb per linear foot.)

Internal Pressure

1. Adding 85 psi (*see* Table 1) for water hammer allowance to the 100-psi

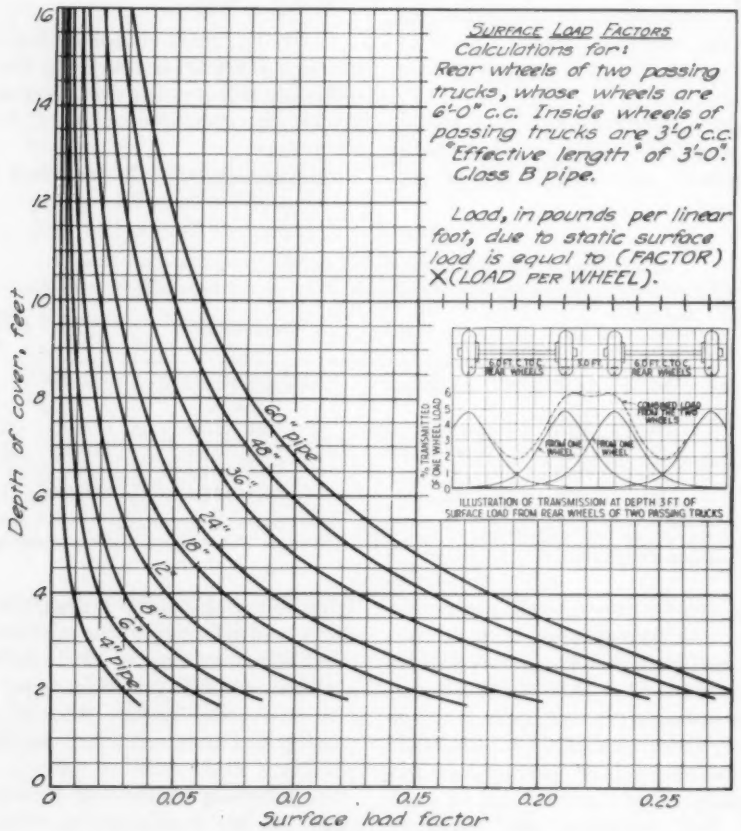

Fig. 9. Surface Load Factors for Two Passing Trucks

Chart shown is Fig. 9 of ASA A21.1 (p. 14). Two passing trucks are assumed, with rear axles over the pipe at the same time, adjacent wheels of the two trucks 3 ft apart (center to center), and load on each rear wheel 9,000 lb. To the load thus obtained, 50 per cent is added for impact of the moving trucks over a road or trench surface assumed to be rough.

working pressure, and multiplying by the factor of safety of 2½, the internal pressure is found to be 463 psi. (If hammer allowance is not made, the pressure, corrected for safety, is 250 psi.)

2. Having determined the total external load (step 6 above) and the total internal pressure, it is now possible to apply the load-pressure thickness nomogram shown in Fig. 8. The net pipe wall thickness with truck load

TABLE 8

*Percentage of Calculated Truck Load Used for Various Field Conditions**

Nominal Diam. of Pipe *in.*	2½–3½-ft Cover		4–7-ft Cover		8–10-ft Cover		More Than 10-ft cover	
	Laying Conditions							
	A, B, E, F	C & D	A, B, E, F	C & D	A, B, E, F	C & D	A, B, E, F	C & D
3–12	100	78	100	84	100	90	100	95
14	92	78	100	84	100	90	100	95
16	88	78	95	84	100	90	100	95
18	85	78	90	84	100	90	100	95
20	83	78	90	84	95	90	100	95
24–30	81	78	85	84	95	90	100	95
36–60	80	78	85	84	90	90	100	95

* From Table 3, ASA A21.1.

TABLE 9

*Ratios to Three-Edge Bearing**

Nominal Diam. of Pipe *in.*	Laying Conditions					
	A	B	C	D	E	F
3	1.15	1.28	0.19	0.82	1.50	1.74
4	1.15	1.29	0.22	0.82	1.50	1.75
6	1.15	1.32	0.31	0.83	1.50	1.78
8	1.15	1.34	0.40	0.84	1.50	1.80
10	1.15	1.36	0.50	0.86	1.50	1.83
12	1.15	1.38	0.60	0.88	1.50	1.85
14	1.15	1.41	0.67	0.91	1.50	1.88
16	1.15	1.43	0.73	0.95	1.50	1.90
18	1.15	1.45	0.78	0.98	1.50	1.93
20	1.15	1.47	0.81	1.01	1.50	1.95
24	1.15	1.52	0.87	1.07	1.50	2.00
30	1.15	1.58	0.93	1.14	1.50	2.07
36	1.15	1.64	0.96	1.19	1.50	2.14
42	1.15	1.69	0.98	1.23	1.50	2.20
48	1.15	1.72	0.99	1.25	1.50	2.25
54	1.15	1.75	0.99	1.28	1.50	2.29
60	1.15	1.77	1.00	1.31	1.50	2.31

* From Fig. 3, ASA A21.1.

and without water hammer is 0.51 in. Adding to this 0.08 in. as the specified constant corrosion allowance, and 0.08 in. for foundry tolerance (as determined from Table 2), the calculated design thickness is 0.67 in. Referring to Table 3 this is found to correspond to Class 22.

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American Water Works Association

Tentative

Standard Specifications

for

DIMENSIONS FOR STEEL WATER PIPE FITTINGS

The American Water Works Association has adopted and promulgates these "Dimensions for Steel Water Pipe Fittings." They are to be considered as dimensional only. They are based upon the best known experience and are intended for use under normal conditions. They are not designed for unqualified use under all conditions and the advisability of use of the dimensions herein specified for any installation must be subjected to review by the engineer responsible for the construction in the particular locality concerned.

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Tentative Standard Specifications for

Dimensions for Steel Water Pipe Fittings

Scope. This standard covers overall dimensions for steel water pipe fittings. It is not intended to cover thickness, shape, welding, attachments, and tolerance of fittings.

Size. The "size" of the fittings in the following tables is identified by the corresponding "nominal pipe size," and may be either inside diameter or outside diameter, depending upon the purchaser's requirements.

TABLE 1 (*See Fig. 1*)

Dimensions for Steel Water Pipe Fittings, Class 125 for Service in Piping Systems in Water Works or Industrial Plants *

(*All Dimensions in Inches*)

Nominal Diam.	Tee		Cross *A+A*	Lateral (45–75 deg)		True Wye (45 deg)		Elbows					Reducer Cone *G*
	Run *A+A*	Outlet *A*		Run *D*	Outlet *E*	*E'*	*F'*	Two-Piece (0–30 deg) *C*	Three-Piece (30–60 deg) *C*	Four-Piece (60–90 deg) *A*	Four-Piece Long-Radius *B*	Smooth (90 deg) *T*	
4	13	6.5	13	15	12	6.5	3	4	4	6.5	9	6	7
6	16	8	16	18	14.5	8	3.5	5	5	8	11.5	9	9
8	18	9	18	22	17.5	9	4.5	5.5	5.5	9	14	12	11
10	22	11	22	25.5	20.5	11	5	6.5	6.5	11	16.5	15	12
12	24	12	24	30	24.5	12	5.5	7.5	7.5	12	19	18	14
14	28	14	28	33	27	14	6	7.5	7.5	14	21.5	21	16
16	30	15	30	36.5	30	15	6.5	8	8	15	24	24	18
18	33	16.5	33	39	32	16.5	7	8.5	8.5	16.5	26.5	27	19
20	36	18	36	43	35	18	8	9.5	9.5	18	29	30	20
22	40	20	40	46	37.5			10	10	20	31.5	33	22
24	44	22	44	49.5	40.5	22	9	11	11	22	34	36	24
30	50	25	50	59	49	25	10	15	15	25	41.5	45	30
36	56	28	56	84†	60†			18	18	28	49		36
42	62	31	62	95†	69†			21	21	31	56.5		42
48	68	34	68	104†	77†			24	24	34	64		48

* All dimensions shown conform to the dimensions given in ASA B16.1 for cast-iron pipe flanges and flanged fittings.

† These fittings shall conform to AWWA Standard Specifications for Cast-Iron Pressure Fittings, C100. If these are to be used with mechanical couplings, add 8 in. to each end for diameters up to 30 in.; 10 in. to each end for diameters of 30 in. and more. For laterals, use the dimensions given in Table 2. Flanges to be ring type.

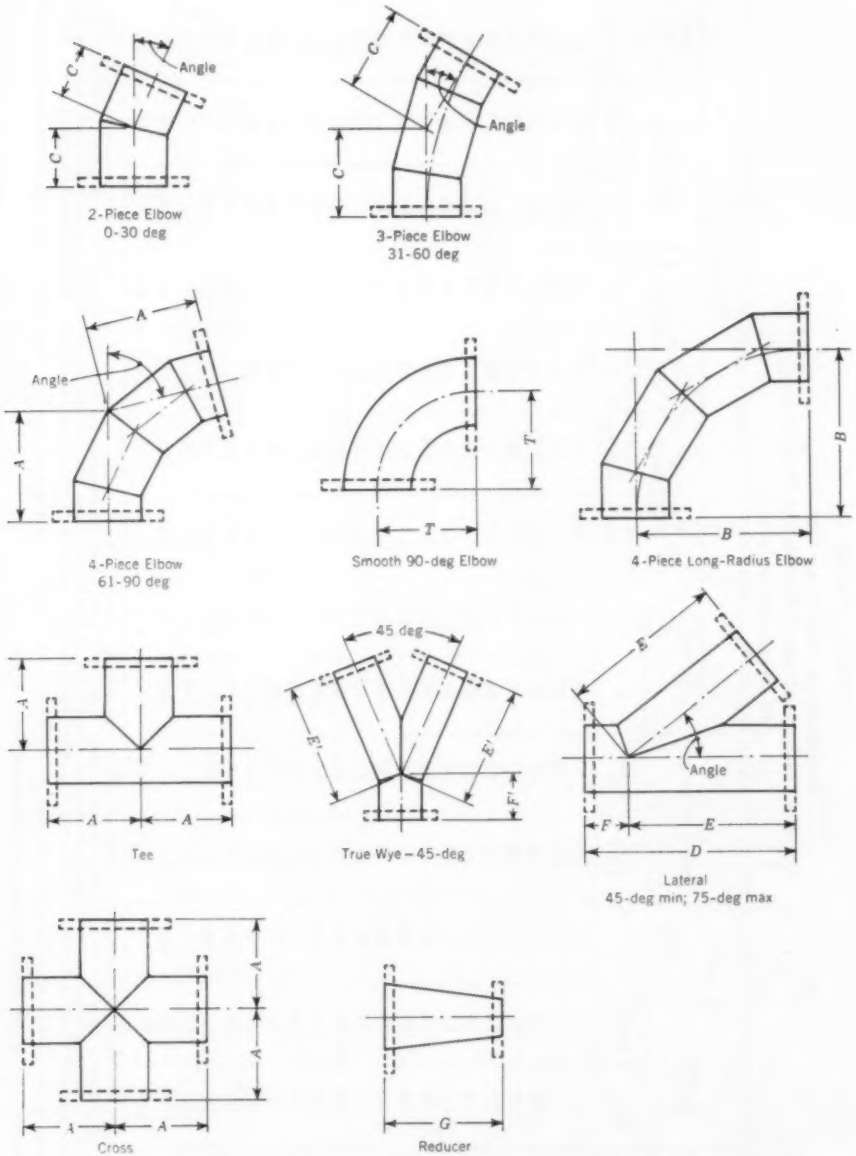


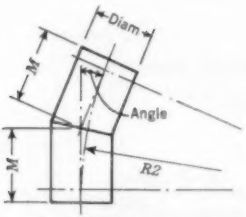
Fig. 1. Steel Water Pipe Fittings, Class 125, for Service in Piping Systems in Water Works or Industrial Plants

TABLE 2 (*See Fig. 2*)

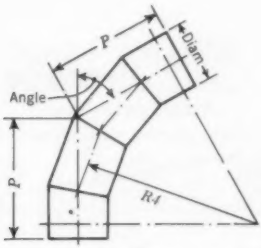
Dimensions for Steel Water Pipe Fittings
*for Service in Transmission or Distribution Main Systems**
(*All Dimensions in Inches*)

Nominal Diam.	Tee		Cross	Lateral (30-deg Minimum)		True Wye (45 deg)		Elbows						
								Two-Piece (0–22.5 deg)		Three-Piece (22.5–45 deg)		Four-Piece (45–67.5 deg)		Five-Piece (67.5–90 deg)
	Run $J+J$	Outlet J	$J+J$	Run $X+Y$	Outlet Y	Y	X	M	$R2$	N	$R3$	P	$R4$	Q or $R5$
4	22	11	22	38	30	30	8	9	45	12	30	15	23	19
6	24	12	24	42	32	32	10	9	45	13	32	16	24	21
8	26	13	26	46	36	36	10	10	50	14	35	18	27	24
10	28	14	28	50	40	40	10	10	50	15	37	20	30	26
12	30	15	30	54	44	44	10	11	55	16	40	22	33	29
14	32	16	32	58	48	48	10	11	55	17	42	24	36	31
16	34	17	34	62	52	52	10	12	60	18	44	26	40	34
18	36	18	36	66	56	56	10	12	60	19	47	27	41	36
20	38	19	38	72	60	60	12	13	65	20	49	28	42	39
22	40	20	40	78	66	66	12	13	65	21	51	30	45	41
24	42	21	42	84	72	72	12	14	70	22	54	32	48	44
30	60	30	60	96	84	84	12	15	75	25	61	37	51	51
36	66	33	66	110	96	96	14	16	80	27	66	40	60	56
42	72	36	72	124	108	108	16	17	85	28	69	42	63	59
48	78	39	78	140	120	120	20	17	85	29	71	44	66	61
54	84	42	84	150	130	130	20	18	90	30	74	46	69	63
60	90	45	90	168	144	144	24	18	90	31	76	48	72	65
66	96	48	96	180	156	156	24	19	95	33	81	50	75	67
72	102	51	102	192	168	168	24	19	95	34	84	52	78	69
78	108	54	108	204	180	180	24	20	100	35	86	54	81	71
84	114	57	114	216	192	192	24	21	105	36	88	56	84	73

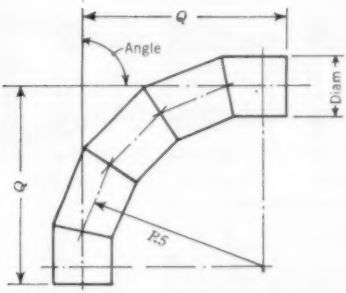
* The dimensions given are recommended standard. Where clearances are restricted, however, other dimensions may be used. Dimensions are based on plain- or flanged-end fittings. Those for tees and crosses agree closely with AWWA Standard Specifications for Cast-Iron Pressure Fittings, C100, exclusive of the bell. Lateral dimensions given cover 30–75 deg angles; for angles greater than 75 deg, use dimension, given for tees, which will allow sufficient clearance for fabrication of reinforcing plates or stiffeners, and welding operations. For angles less than 30 deg, use 30-deg lateral plus an elbow to fit. Elbow radiuses are approximate. Minimum distance between welded girth joints for pipe sizes of 4–30 in. is $4 + 8\,\frac{(D-4)}{26}$. These values, therefore, vary from 4 in. for 4-in. diameters to 12 in. for 30-in. diameters. For all pipe sizes over 30 in., the minimum distance is 12 in.



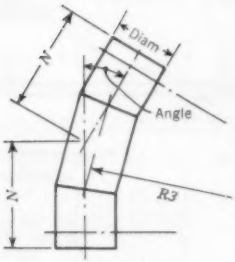
2-Piece Elbow
0-22.5 deg



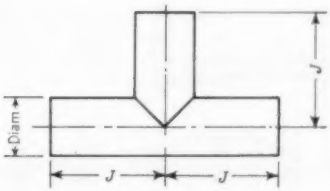
4-Piece Elbow
45-67.5 deg



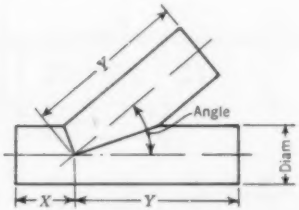
5-Piece Elbow
67.5-90 deg



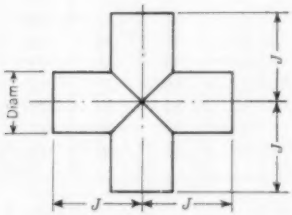
3-Piece Elbow
22.5-45 deg



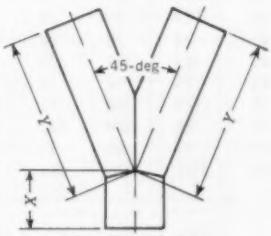
Tee



Lateral
45 deg min; 75-deg max



Cross



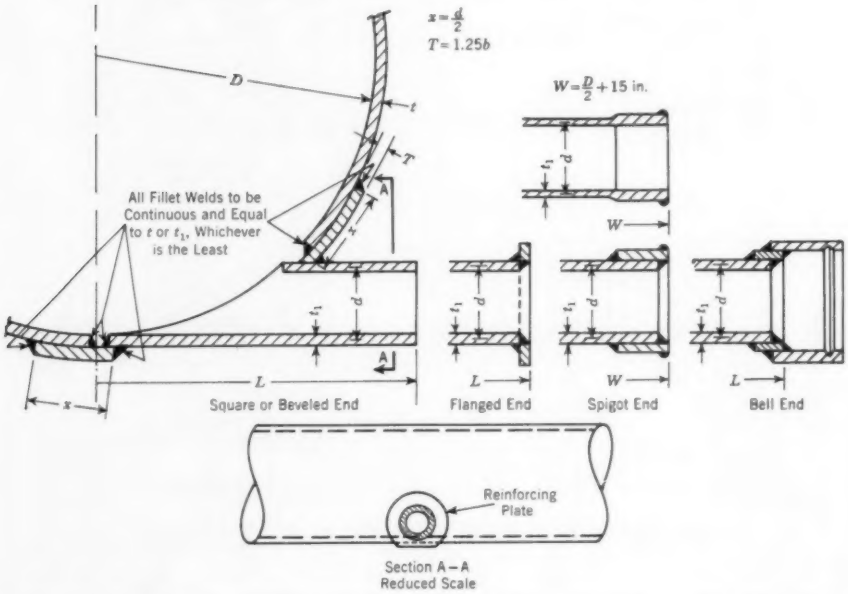
True Wye – 45-deg

Fig. 2. Steel Water Pipe Fittings for Service in Transmission or Distribution Main Systems

D is the inside diameter of the pipeline, and d is the nominal diameter of the blowoff connection. L may be varied slightly to suit specific requirements. For details of flanges, see AWWA C207. For details of bells and spigots, see Table 4.

TABLE 3

Dimensions for Welded-Steel Tangent-Type Blowoffs



D, in.	*d* = 4 in.	*d* = 6 in.	*d* = 8 in.	*d* = 10 in.	*d* = 12 in.	*D*, in.	*d* = 4 in.	*d* = 6 in.	*d* = 8 in.	*d* = 10 in.	*d* = 12 in.
	L—in.						*L*—in.				
24	19	21	22	22	22	63	28	31	34	35	37
27	20	22	24	24	24	66	28	31	34	36	38
30	21	23	25	25	25	69	29	32	35	37	39
33	22	24	26	26	26	72	29	33	35	38	39
36	22	25	27	28	28	75	30	33	36	39	40
39	23	26	28	29	29	78	30	34	37	39	41
42	24	26	28	30	30	81	31	34	37	40	41
45	24	27	29	31	31	84	31	35	38	40	42
48	25	28	30	32	33	87	31	35	38	41	43
51	25	28	31	32	33	90	32	36	39	42	43
54	26	29	32	33	34	93	32	36	39	42	44
57	27	30	33	34	35	96	33	37	40	43	45
60	27	30	33	35	36	99	33	37	40	43	45

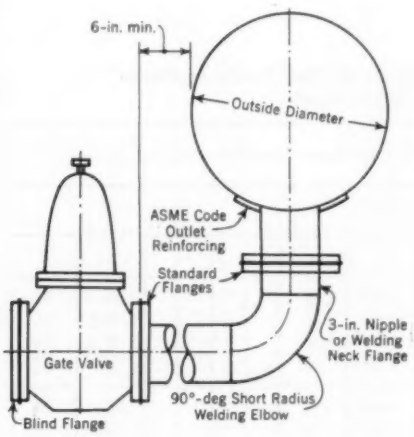


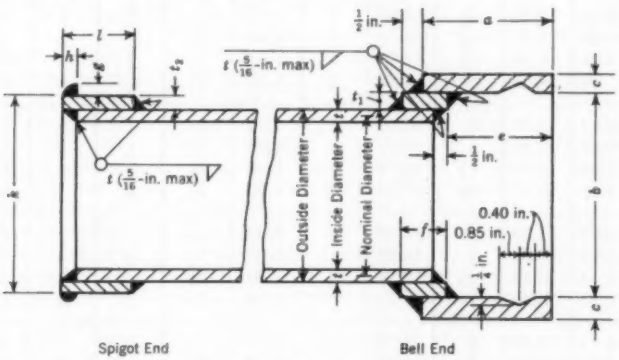
TABLE 3 ALTERNATE

Dimensions for Welded-Steel Vertical-Type Blowoffs

Nominal Pipe Size* in.	Outside Diameter in.	Pipe Thickness in.
4	4.5	0.337
6	6.63	0.432
8	8.63	0.322
12	12.75	0.375

* The 4-in. and 6-in. pipe is extra strength; the 8-in. and 12-in. is standard.

TABLE 4—*Dimensions for Steel Pipe Ends for Connections to Cast-Iron Bell-and-Spigot Pipe* *



Cast-Iron Pipe Dimensions—in.			Steel Bell-and-Spigot Dimensions—in.								
			Bell End					Spigot End			
Nominal Diam.	Thickness	Outside Diam.	*a*	*b*	*c*	*e*	*f*	*g*	*h*	*k*	*l*
3	0.37–0.45	3.80	5½	4.60	¾	3.50	2½	3/16	⅜	3¾	5
	0.46–0.53	3.96	5½	4.76	¾	3.50	2½	3/16	⅜	4	5
4	0.40–0.45	4.80	5½	5.60	¾	3.50	2½	3/16	⅜	4¾	5
	0.46–0.55	5.00	5½	5.80	¾	3.50	2½	3/16	⅜	5	5
6	0.43–0.50	6.90	5½	7.70	¾	3.50	2½	3/16	⅜	6¾	5
	0.51–0.60	7.10	5½	7.90	¾	3.50	2½	3/16	⅜	7⅛	5
	0.61–0.66	7.22	6	8.02	¾	4.00	2½	3/16	⅜	7½	5½
	0.67–0.74	7.38	6	8.18	¾	4.00	2½	3/16	⅜	7⅞	5½

* Dimensions are based on American Standard Specifications for Cast-Iron Pit-Cast Pipe for Water or Other Liquids—ASA A21.2 (AWWA C102) and AWWA C100.

TABLE 4 (*contd.*)—*Dimensions for Steel Pipe Ends for Connections to Cast-Iron Bell-and-Spigot Pipe* *

Cast-Iron Pipe Dimensions—*in.*			Steel Bell-and-Spigot Dimensions—*in.*								
			Bell End					Spigot End			
Nominal Diam.	Thickness	Outside Diam.	*a*	*b*	*c*	*e*	*f*	*g*	*h*	*k*	*l*
8	0.46–0.57	9.05	6	9.85	1	4.00	2½	¼	½	9	5½
	0.58–0.70	9.30	6	10.10	1	4.00	2½	¼	½	9 5/16	5½
	0.71–0.76	9.42	6	10.22	1	4.00	2½	¼	½	9 7/16	5½
	0.77–0.85	9.60	6	10.40	1	4.00	2½	¼	½	9 5/8	5½
10	0.50–0.60	11.10	6	11.90	1	4.00	2½	¼	½	11⅛	5½
	0.61–0.75	11.40	6	12.20	1	4.00	2½	¼	½	11⅜	5½
	0.76–0.85	11.60	6½	12.40	1	4.50	2½	¼	½	11⅝	6
	0.86–0.97	11.84	6½	12.64	1	4.50	2½	¼	½	11⅞	6
12	0.54–0.65	13.20	6½	14.00	1	4.00	2½	¼	½	13¼	6
	0.66–0.80	13.50	6½	14.30	1	4.00	2½	¼	½	13½	6
	0.81–0.94	13.78	6½	14.58	1	4.50	2½	¼	½	13¾	6
	0.95–1.09	14.08	6½	14.88	1	4.50	2½	¼	½	14⅛	6
14	0.54–0.62	15.30	6	16.10	1	4.00	2½	¼	½	15 5/16	5½
	0.63–0.87	15.65	6	16.45	1	4.00	2½	¼	½	15⅝	5½
	0.88–1.04	15.98	6½	16.78	1	4.50	2½	¼	½	16	6
	1.05–1.21	16.32	6½	17.12	1	4.50	2½	¼	½	16 5/16	6
16	0.58–0.67	17.40	6	18.40	1	4.00	2½	¼	½	17⅜	5½
	0.68–0.95	17.80	6	18.80	1	4.00	2½	¼	½	17¾	5½
	0.96–1.13	18.16	6½	18.96	1	4.50	2½	¼	½	18 3/16	6
	1.14–1.32	18.54	6½	19.34	1	4.50	2½	¼	½	18½	6
18	0.63–0.72	19.50	6	20.50	1	4.00	2½	¼	½	19½	5½
	0.73–1.01	19.92	6	20.92	1	4.00	2½	¼	½	19 15/16	5½
	1.02–1.22	20.34	6½	21.14	1	4.50	2½	¼	½	20 5/16	6
	1.23–1.44	20.78	6½	21.58	1	4.50	2½	¼	½	20¾	6
20	0.66–0.82	21.60	6	22.60	1	4.00	2½	¼	½	21⅝	5½
	0.83–1.08	22.06	6	23.06	1	4.00	2½	¼	½	22 1/16	5½
	1.09–1.32	22.54	6½	23.34	1	4.50	2½	¼	½	22½	6
	1.33–1.56	23.02	6½	23.82	1	4.50	2½	¼	½	23	6

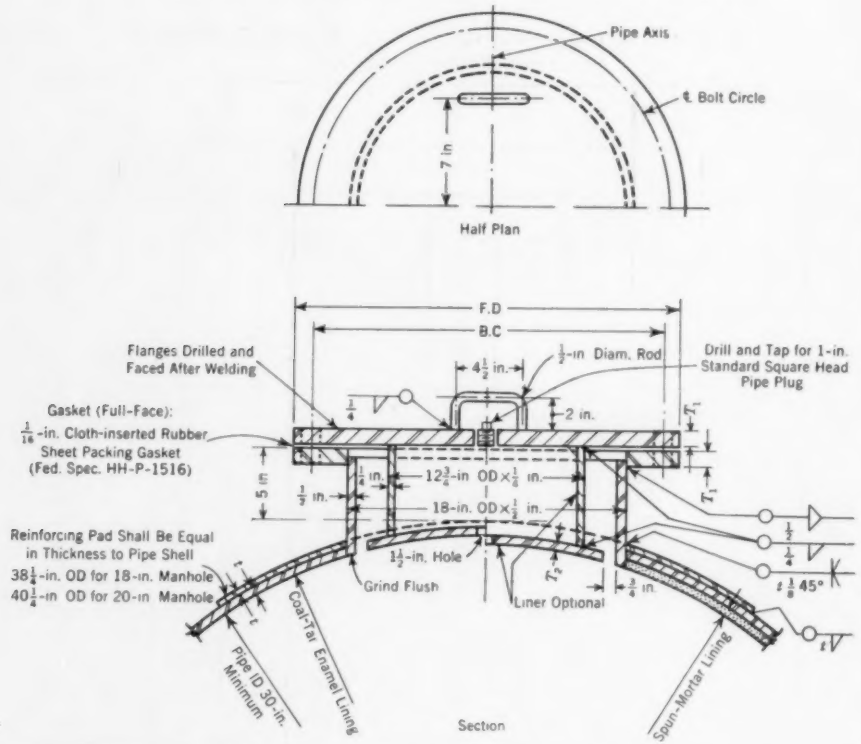
* Dimensions are based on American Standard Specifications for Cast-Iron Pit-Cast Pipe for Water or Other Liquids—ASA A21.2 (AWWA C102) and AWWA C100.

TABLE 4 (*contd.*)—*Dimensions for Steel Pipe Ends for Connections to Cast-Iron Bell-and-Spigot Pipe**

Cast-Iron Pipe Dimensions—*in.*			Steel Bell-and-Spigot Dimensions—*in.*								
			Bell End					Spigot End			
Nominal Diam.	Thickness	Outside Daim.	*a*	*b*	*c*	*e*	*f*	*g*	*h*	*k*	*l*
24	0.74–0.92	25.80	6	26.80	1	4.00	2½	¼	½	25⅜	5½
	0.93–1.21	26.32	6	27.32	1	4.00	2½	¼	½	26 5/16	5½
	1.22–1.50	26.90	7	27.90	1	5.00	2½	¼	½	26⅞	6½
	1.51–1.93	27.76	7	28.56	1	5.00	2½	¼	½	27¾	6½
30	0.87–0.92	31.74	6½	32.74	1	4.50	2½	¼	½	31¾	6
	0.93–1.05	32.00	6½	33.00	1	4.50	2½	¼	½	32	6
	1.06–1.18	32.40	6½	33.40	1	4.50	2½	¼	½	32⅜	6
	1.19–1.42	32.74	6½	33.74	1	4.50	2½	¼	½	32¾	6
	1.43–1.60	33.10	7	34.10	1	5.00	2½	¼	½	33⅛	6½
	1.61–1.78	33.46	7	34.46	1	5.00	2½	¼	½	33½	6½
36	0.97–1.03	37.96	6½	38.96	1	4.50	2½	¼	½	38	6
	1.04–1.20	38.30	6½	39.30	1	4.50	2½	¼	½	38¼	6
	1.21–1.40	38.70	6½	39.70	1	4.50	2½	¼	½	38⅝	6
	1.41–1.63	39.16	6½	40.16	1	4.50	2½	¼	½	39⅛	6
	1.64–1.85	39.60	7	40.60	1	5.00	2½	¼	½	39⅝	6½
	1.86–2.07	40.04	7	41.04	1	5.00	2½	¼	½	40	6½
42	1.07–1.15	44.20	7	45.20	1	5.00	2½	¼	½	44¼	6½
	1.16–1.30	44.50	7	45.50	1	5.00	2½	¼	½	44½	6½
	1.31–1.57	45.10	7	46.10	1	5.00	2½	¼	½	45⅛	6½
	1.58–1.84	45.58	7	46.58	1	5.00	2½	¼	½	45⅝	6½
48	1.18–1.30	50.50	7	51.50	1	5.00	2½	¼	½	50½	6½
	1.31–1.45	50.80	7	51.80	1	5.00	2½	¼	½	50¾	6½
	1.46–1.72	51.40	7	52.40	1	5.00	2½	¼	½	51⅜	6½
	1.73–2.04	51.98	7	52.98	1	5.00	2½	¼	½	52	6½
54	1.30–1.38	56.66	7½	57.66	1	5.50	2½	¼	½	56⅝	7
	1.39–1.60	57.10	7½	58.10	1	5.50	2½	¼	½	57⅛	7
	1.61–1.89	57.80	7½	58.80	1	5.50	2½	¼	½	57¾	7
	1.90–2.25	58.40	7½	59.40	1	5.50	2½	¼	½	58⅜	7
60	1.39–1.45	62.80	7½	63.80	1	5.50	2½	¼	½	62¾	7
	1.46–1.75	63.40	7½	64.40	1	5.50	2½	¼	½	63⅜	7
	1.76–2.15	64.20	7½	65.20	1	5.50	2½	¼	½	64¼	7
	2.16–2.46	64.82	7½	65.82	1	5.50	2½	¼	½	64⅞	7

* Dimensions are based on American Standard Specifications for Cast-Iron Pit-Cast Pipe for Water or Other Liquids—ASA A21.2 (AWWA C102) and AWWA C100.

TABLE 5—*Dimensions for 18-in. and 20-in. Welded-Steel Manholes**



Working Pressure† *psi*	F.D. *in.*	B.C. *in.*	T_1 *in.*	Number of Bolts‡	Bolt Size—*in.*	Bolt Hole Diam.—*in.*
18-in. Manhole						
150	25	22 3/4	1 1/16	16	1 1/8	1 1/4
175	25	22 3/4	1 1/16	16	1 1/8	1 1/4
225	28	24 3/4	1 3/8	24	1 1/4	1 3/8
250	28	24 3/4	1 1/2	24	1 1/4	1 3/8
300	28	24 3/4	1 5/8	24	1 1/4	1 3/8
20-in. Manhole						
150	27 1/2	25	1 1/8	20	1 1/8	1 1/4
175	27 1/2	25	1 1/8	20	1 1/8	1 1/4
225	30 1/2	27	1 3/8	24	1 1/4	1 3/8
250	30 1/2	27	1 9/16	24	1 1/4	1 3/8
300	30 1/2	27	1 3/4	24	1 1/4	1 3/8

* T_2 is 1/4 in. or may be coupon cut from main cylinder. For pipe slopes greater than 5 deg, the riser pipe is to be set vertical. The inside face of the manhole is to be flush with the finished interior pipe surface. F.D. stands for flange dimension, B.C. for bolt circle.

† Working pressure is to be stamped on manhole by the fabricator with 1/4-in. dies.

‡ Hexagonal bolts and nuts are to be used.

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All water meters are fine instruments. Any fine instrument loses accuracy as it wears. Because of this fact, written between the lines of any water meter price are two items you can't read for many years. Yet they are vital in choosing the lowest "total price."

First, as a meter loses accuracy, it gives away revenue. Before its first overhaul it may give away twice as much money as it seems to save in its "low bid" original price.

Secondly, to keep the meter accurate you may have higher repair costs over the years . . . or premature scrap-and-replace costs. Good meters often vary widely in this respect.

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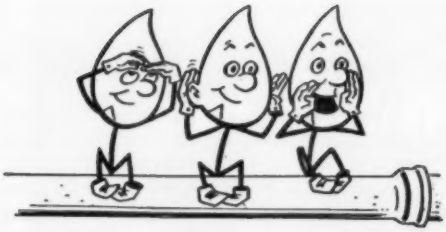

Percolation and Runoff

The fluoridation furor continues to keep our mailbox full to overflowing with press clippings, which, by their bulk if not their novelty or variety, rate some recognition. Actually, except for their datelines, the stories of the present would be difficult to distinguish from those of 1 or 2, or even 5 years ago. The same elements in each community, using the same basic facts and supported by the same medical, dental, and public health authorities, promote fluoridation, and the same other elements dispute the same facts and raise the same legal, religious, political, moral, and even scientific questions as have been posed since the beginning. Meanwhile, though the list of fluoridating communities has suffered a few losses—such as Amsterdam, N.Y., which last March discontinued fluoridation after 3 years, because some councilmen "were not convinced the program was doing the job intended"—it continues to gain many more than it loses, and the addition of Chicago's four million beginning on May 1 has made the gain in population served almost overwhelming.

Among the highlights from the latest batch of clippings has been the news from Ontario and New Brunswick, where the courts have held that fluoridation is not permissible under present provincial law. Work on enabling legislation is already under way and, particularly in Ontario, is being pushed hard. Profiting by these experiences, the Greater Winnipeg Water District obtained its enabling legislation first, getting the Manitoba legislature to amend its District Act to permit fluoridation if the councils of five of the nine member municipalities approved and if the populations of the approving communities constituted at least 85 per cent of the total. Those rather rugged requirements were met at the end of May, and it was expected that the district would approve fluoridation at its June meeting. Meanwhile, Brandon, Man., has become one of the first communities in the province to fluoridate, and, in Alberta, four towns have already requested authority to proceed. In the lead, though, is Saskatchewan, with 12 per cent of its population receiving fluoridated water.

Back in Ontario, the legal setback, which postponed action not only in metropolitan Toronto, but in Hamilton and Ottawa as well, now seems to have caused still another chemical reaction. Flushed with its success in "stopping" fluoridation, the Pure Water Universal Plea Organization has now announced that it will immediately launch a campaign to block the

(*Continued on page 36 P&R*)

(Continued from page 35 P&R)

use of chlorine in Toronto's water supply.

But all the excitement hasn't moved across the northern border. Throughout the US, the same forces are coming into conflict in town after town—and in each new one the old arguments are somehow rediscovered. Particularly in the New York metropolitan area now, the story is being unfolded in community after community with scarcely a line omitted and certainly no sign of cosmopolitan coolness. In New York City itself, Mayor Wagner is in favor of fluoridation, but his water commissioner is strongly opposed, and there the matter rests—rather uncomfortably. In Westchester, some communities do, some don't, and at least one claims to have had it done unbeknownst, unwanted, and for 5 years. On Long Island and in northern New Jersey, the picture is much the same with more don't's than do's and, therefore, furor.

In AWWA, meanwhile, all has not been exactly furorless either—with one section asking for a restatement of policy in favor of fluoridation and another, against, while a third feels that the Association should make its own investigation of the pros and cons. Most members, though, have found it wisest to disclaim medical or dental competence and to indicate merely their ability to handle the mechanics of fluoridation once the decision is made. And it was on that basis that the Board of Directors at St. Louis last May once more reaffirmed AWWA's original policy statement:

In communities where a strong public demand has developed and the procedure has the full approval of the local medical and dental societies, the local and state health authorities, and others responsible for the communal health, water departments or companies may properly participate in a program of fluoridation of public water supplies.

Recommendations for fluoridation are the prerogative of the dental, medical, and public health groups. When the proper authorities approve the treatment, it then becomes the function of the water works utility and industry to follow through willingly and intelligently where proper controls are assured.

Furor, indeed, but what a lot of noise is made!

Shahmoon Industries, Inc., is the new name of Warren Foundry & Pipe Corp., New York. S. E. Shahmoon is the firm's president and chairman of the board. The company will apply the former name to one of its divisions, the "Warren Foundry & Pipe Div."

Our hydrantana continues to swell. No sooner had we closed the books on a banner year of fire plug fact and fable (*see* April issue, p. 54 P&R) than we had to open them again for a series of stories that actually offered something new. At Dallas, Tex., in March, for instance, a report that a driver, failing to make a turn, had sheared off a hydrant was hardly startling, but when he returned to the scene and hauled the fallen hydrant away with him, it was undoubtedly the first such attempt at suppressing mighty heavy evidence. Then at Corning, N.Y., in May, the dissatisfaction of a "little old lady" with the color scheme of the hydrant outside her home hardly rated headlines, but she added color to the story when she camouflaged the plug in delicate shades of pink and gray and managed to maintain the decor for at least a little while before the public works superintendent noticed it and decided it was too, too illegal to per-

(Continued on page 38 P&R)

New Recarbonator Principles

provide GREATER EFFICIENCY in MUCH LESS SPACE

Revolutionary new water stabilization system with control panel can be designed to meet your requirements.

Proved in actual service, this development by INFILCO engineers offers you many advantages in CO_2 generation and distribution. Since the burner and combustion chamber are located in the top of the "downcomer," the lower portion of which is immersed in the recarbonation basin water itself, costly cooling apparatus is eliminated. There are no exposed hot gas lines.

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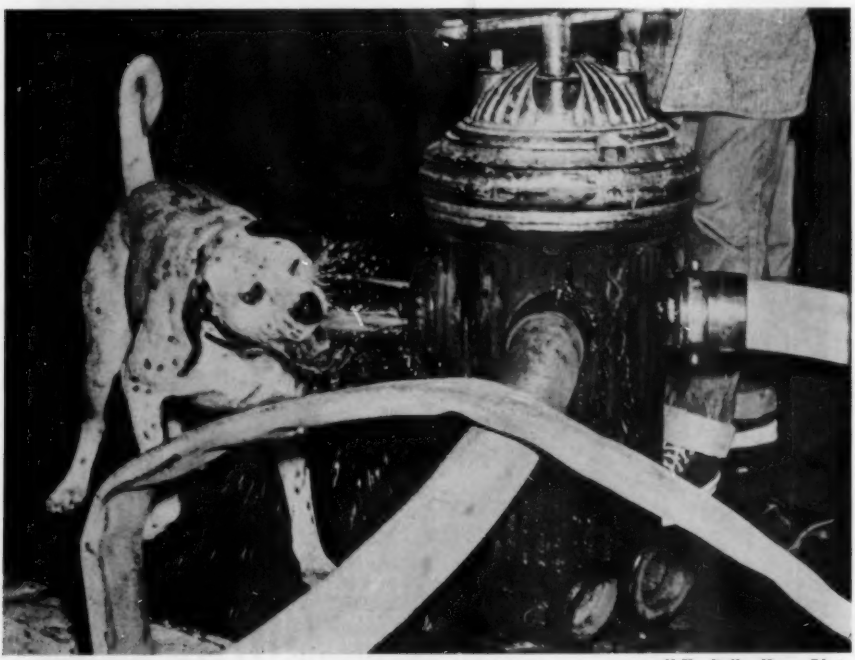
(Continued from page 36 P&R)

Atlanta's 10,000th Hydrant With AWWA's 77th President

mit. And at Atlanta, Ga., the chrome-plated hydrant which appeared one day last March might have been just another shiny plug had it not been built especially for the occasion and presented by the Mueller Co. to the water department general manager, Paul Weir, as the 10,000th of Atlanta's hydrants (see cut at left).

All new and all news, however, was the story from New York City in May, when the historic photograph (below) of a hydrant taking revenge on one of its daily desecrators was made. Actually the self-protection was not built into the hydrant by the manufacturers, but drilled in after installation, primarily to give New York children the relief of a summer spraying without necessitating use of a full hydrant stream. Actually, too, it isn't really

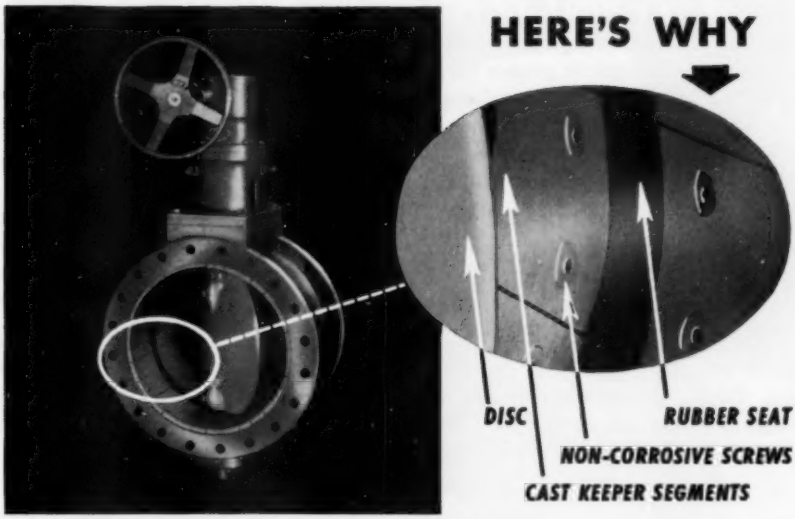


Hydrant Wets Dog

N.Y. Daily News Photo

(Continued on page 40 P&R)

125# Valves...drop tight!



Will not "freeze" from corrosion— *Corrosion can't hinder operation.* Disc edges are chrome plated or stainless steel and close into a thick rubber seat. These features plus self lubricated bearings and stainless steel shafting make it impossible for valves to "corrode tight" either open or shut.

Full 90° closure into Rubber Seat— The disc is slightly larger than the inside of the rubber seat and thus actually displaces the rubber when closed. The ratio of seat thickness to disc indentation insures drop-tight shut off and avoids permanent set. The seats are secured in the valve body by removable keeper segments.

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RUBBER SEAT Butterfly Valves

Henry Pratt Company, 2222 S. Halsted St., Chicago 8, Ill. Representatives in principal cities

(Continued from page 38 P&R)

revenge in Spotty's case. for he, as mascot of Engine Co. No. 11, would never think of abusing a hydrant. That undoubtedly is why the hydrant is merely giving him a playful squirt in the eye while catering to his thirst. Looking at the picture in proper focus, though, we might well see and cheer the fact that the canine world has at last discovered at which end of the water system the hydrant properly belongs.

James D. Bailey has been appointed administrative assistant to the sales manager at Mueller Co., Decatur, Ill. He was formerly credit manager.

John H. Hoff has been appointed Tulsa, Okla., sales representative for hot-enamel products of Reilly Tar & Chemical Corp.

May issue missing? If you have been receiving your copies of the JOURNAL regularly but did not get the May number (or got a singed or water-logged copy), the reason probably is that it was among those in a fire-ridden railroad car at Huntingdon, Pa. Please notify JOURNAL AWWA, 2 Park Ave., New York 16, N.Y., and we will forward a copy to you.

Noel S. Chamberlin, for more than 16 years a sanitary chemist with Wallace & Tiernan Inc., has left to join the consulting firm of Havens & Emerson, New York, as a chemical engineer.

Elmer L. Nordness has been named superintendent of water works and sewerage at Madison, Wis. He succeeds the late Leon A. Smith, whose assistant he was for 20 years.

(Continued on page 42 P&R)

ELEVATED STEEL TANKS

● Elevated Steel Tanks for water supply, ranging from 5,000 to 2,000,000 gallons—ranging from standard hemispherical self-supporting bottom to spherical tank on tubular tower.

Correctly built in accordance with AWWA specifications. Send us your inquiry, stating capacity, height to bottom and location. Established 1854. Write for *Tank Talks*.

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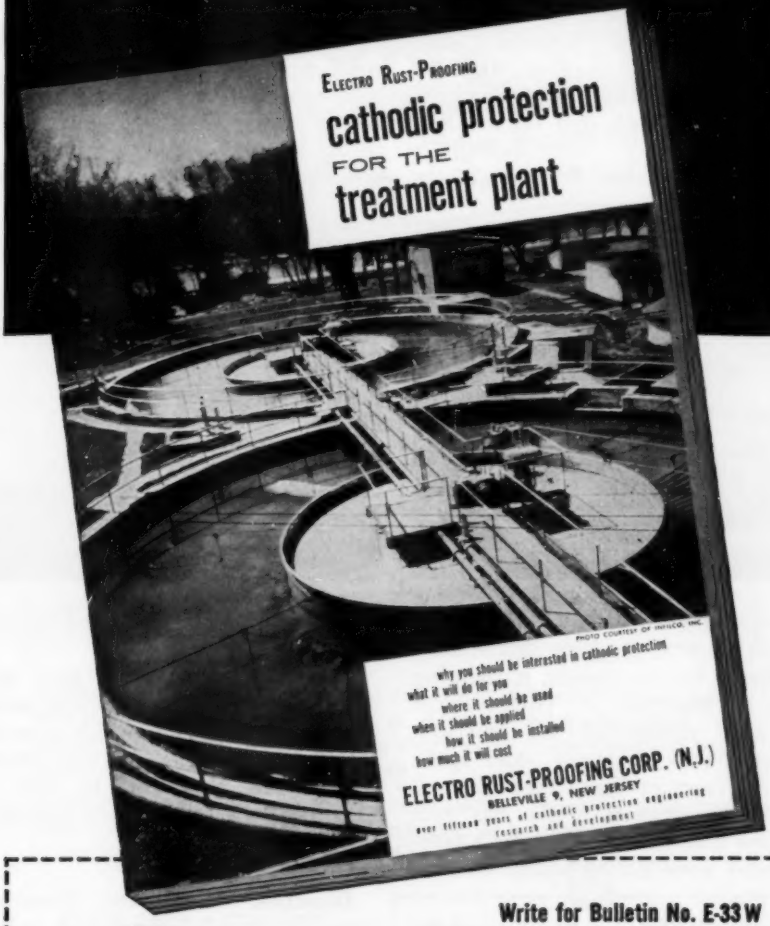
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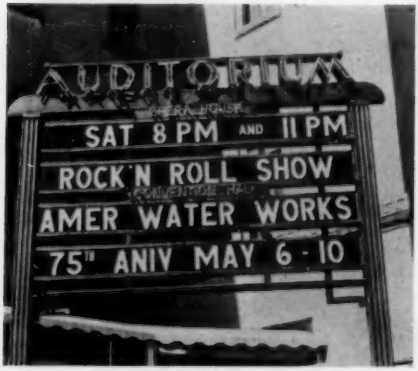
E-33

(Continued from page 40 P&R)

St. Louis views not included in last month's roundup of the Diamond Jubilee Conference were the two shown below. That on the left came by blackmail from Fred Maurer of *Water & Sewage Works* with some question about the "technical" program. That on the right, appearances to the contrary notwithstanding, has absolutely no connection with that on the left. On the contrary, withstanding, it shows the life-size Willing Water



owned by the St. Louis County Water Co. on stage at the 75th Anniversary Session of the Conference, introducing John H. Murdoch Jr., vice-president and chief counsel of the American Water Works Company, who spoke on "75 Years of Too Cheap Water." In this incarnation, Willie has eyes and lips that rock 'n' roll, plus a tape-recorded voice full of water wisdom.

Rockwell Mfg. Co. has acquired the assets of Locomotive Finished Materials Co., Atchison, Kan. LFM, which continues under its former management as a major division of Rockwell, has the largest steel foundry and machining facilities west of the Mississippi.

Journal AWWA has received a 1955 Public Interest Award from the National Safety Council. This noncompetitive award is made annually to a number of public information media "for exceptional service to safety." It will be recalled that the JOURNAL devoted a portion of six issues last year to serializing the *AWWA Safety Manual,* prepared by the Committee on Safety Practices. The award, then, is actually in recognition of the outstanding efforts of the members of this committee.

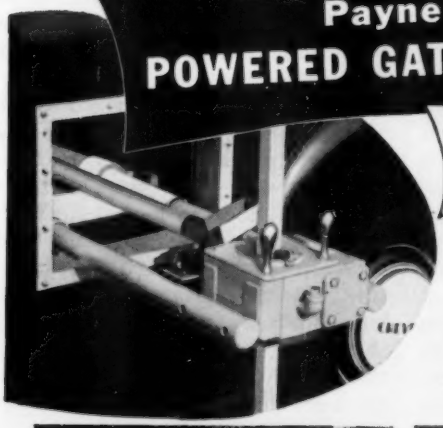
The *Safety Manual* has since been issued as a separate 128-page handbook, designated M3, which is available from the AWWA Order Dept., 2 Park Ave., New York 16, N.Y., at $1.50 per copy (to members paying in advance, $1.20).

Diamond Alkali Co., Cleveland, Ohio, has established two new operating divisions—Electro Chemicals Div., with Frank Chrencik as general manager; and Soda Products Div., with Henry B. Clark as general manager. This completes a functional reorganization begun by the company in 1953.

(Continued on page 44 P&R)

Make every valve a Power Operated Valve!

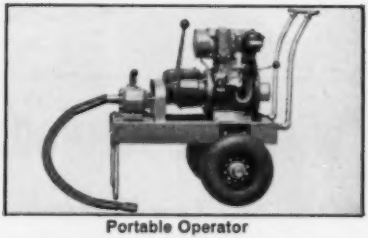
Payne Dean
POWERED GATE-OPERATORS



Save hundreds of man-hours

Cut costs and improve maintenance

Truck-Mounted Operator



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Valves should be inspected, opened and closed at least twice each year. You can easily keep this schedule of recommended maintenance . . . within your budget . . . by making every valve Power-Operated with Payne Dean Powered Gate-Operators.

Truck models operate from truck transmission . . . portable units from 3 to 7½ H. P. . . . operate valves and gates as large as 60". Save time . . . reduce cost . . . improve maintenance.

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(Continued from page 42 P&R)

The more head, the more head—and the more headache! That was the story at Atlanta, Ga., and Youngstown, Ohio, last month, when the water systems involved broke out in suds.

At Atlanta, the whole downtown area was affected when a fountain in Plaza Park so subsudded the area that extra police had to be called out to reroute traffic, and two fire companies were put on defoamation duty. As it turned out, the firemen actually contributed to the upbubblement, for the basic cause was a highly concentrated detergent which some prankster had poured into the fountain. And it wasn't until 2 hr after the foamcoming that water works man J. D. Vaughn was called to the scene to end the ensudsment by shutting off the fountain supply line (J. D., by the way, is not to be confused with J. C. Vaughn, chairman of the AWWA Task Group on Analytical Methods for the Determination of Synthetic Detergents in Water, and prominent member of the AWWA Task Group on Effects of Synthetic Detergents on Water Supplies).

Clean unexpected, too, was the Youngstown flood of suds, coming as it did in the middle of Mrs. Margaret Swory's family wash before she had a chance to add soap to it. No detergent there, though—this was straight pilsener, coming hot and under pressure. Being on tap, of course, the home foam was somewhat simpler to shut off, if a little sticky in the shirtsleeves. And anything but a prank, the lager in the laundry was merely a

(Continued on page 46 P&R)

WHY USE JOHNSON WELL SCREENS?

1. **Less drawdown.**
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3. **Lowest pumping cost per million gallons of water.**

True economy is measured not by first cost alone, but in lowest yearly cost. The JOHNSON WELL SCREEN combines an unmatched record of experience and dependability with greatest strength and durability. It is the finest and most truly economical well screen in the world.

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East Moline, Illinois, installed this 250,000-gallon Horton® Double Ellipsoidal Bottom Elevated Steel Tank to serve as a dependable source of gravity water supply in their municipal water works system. The tank is 96 feet to the bottom capacity line.

For an estimate or information on the above type of tank or standpipes, reservoirs, Waterspheres,® Waterspheroids or radial cone tanks, contact our nearest office below.

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In Canada—HORTON STEEL WORKS, LIMITED, FORT ERIE, ONT.

(*Continued from page 44 P&R*)

fortuitous cross connection, wherein a keg was hooked to the wrong piping. Fortuitous, but not very fortunate, for either Mrs. Swory or the bartender, although her beery linens probably turned out to be less of a headache than his draft hot water.

Now if Mrs. Swory were smart, she'd head for Atlanta; and we'd be glad to take her Youngstown apartment off her hands.

Howard S. Morse has been elected to the newly created position of chairman of the board of Indianapolis Water Co. Succeeding him as president is Thomas W. Moses, formerly Mr. Morse's assistant.

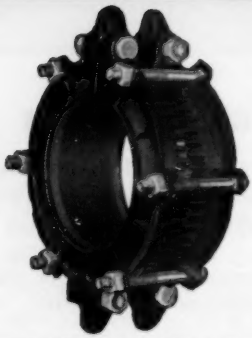
Walter L. Picton, deputy director of Water & Sewerage Industry & Utilities Div., BDSA, Washington, D.C., has been designated by the State Dept. as US specialist on water utilities for a conference of the public utilities working groups that report to the NATO Civil Defense Committee. The meeting will be held in Paris this October.

Edmund J. Laubusch has accepted the position of research chemist with The Chlorine Institute, New York. Mr. Laubusch was formerly a USPHS sanitary engineer officer.

Gerald J. Bayern has joined Barrett Div., Allied Chemical & Dye Corp., New York, as marketing and economic analyst. Mr. Bayern was formerly with Foster D. Snell, Inc., as assistant director of engineering.

(*Continued on page 48 P&R*)

FOR REPAIRING BROKEN MAINS



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(Continued from page 46 P&R)

One Goodell turn deserves another —and another is what this year's winners of AWWA's John M. Goodell prize received, being honored even in their own country. In Los Angeles, on May 8, even before the St. Louis Conference was over, the Board of Water & Power Commissioners adopted the following resolution:

Messrs. Herbert S. Swanson, Herbert J. Chapton, Carleton L. King, Eugene D. Nelson, employees of the Department of Water and Power, and Mr. William J. Wilkinson, retired employee, have for the past 10 years carried on original research on the design of wye branch pipes.

The results of this research work were incorporated in a report entitled, "Design of Wye Branches for Steel Pipe," which has received national recognition as an outstanding contribution to the water works industry.

In recognition of their achievement, the American Water Works Association has awarded these men the John M. Goodell prize for having made the most notable contribution to the science or practice of water works development during the year 1955. Therefore be it

RESOLVED that this Board does hereby express both its sincere appreciation for the service rendered by these men to the water works industry and its great pride in their engineering achievement.

And now it's your turn—the prize-winning paper, which appeared in the June 1955 JOURNAL, pp. 581–630, deserves being turned to!

Charles Bueltman has been promoted to manager of the Industrial Waste Treatment Dept. of The Permutit Co., New York. Mr. Bueltman has been with the company since 1948.

(Continued on page 50 P&R)

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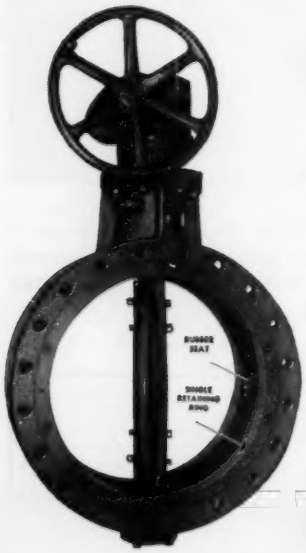

In this design, the seat is of resilient neoprene rubber or pure gum rubber, vulcanized around a steel ring insert, and held in place by a keeper ring. The steel ring, which is "continuous" to eliminate abnormal wearing, increases the firmness of the seat and assures longer service life.

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(*Continued from page 48 P&R*)

Tortoise in the hair was the situation last April, when a number of loggerhead turtles crawled into the water distribution system at Okeechobee, Fla., while the pumps were shut down temporarily to permit installation of some new pipe. What was slow, though, was the flow when the pumps were started again; it wasn't long, actually, before the whole system was completely plugged. That put maintenance crews to work all night in a race to get water back into the system by the time the customers awoke, and in the race, of course, the tortoises had to come out first.

Populous pipes were a problem elsewhere too. In New York City, the consternation was considerable when a brook trout emerged from a water main at fashionable 5th Avenue and 58th Street as a break flooded the area. The fish story involved was one of alewives (the herring, not the *frau*, type) at the reservoir. Having made their septennial pilgrimage to fresh water this year, these herringlike visitors had been clogging the fish screen at the entrance to the aqueduct until hungry brook trout were introduced as a control. The one who got away was either surfeited, after bigger herring, or anxious to see the big city. At any rate, he saw, was seen, and was undoubtedly excellent broiled.

Not broiled, but boiled, might well have been the diagnosis in St. Louis if William F. Power hadn't had the presence of mind to impound the evi-

(*Continued on page 98 P&R*)

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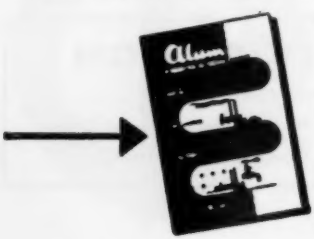

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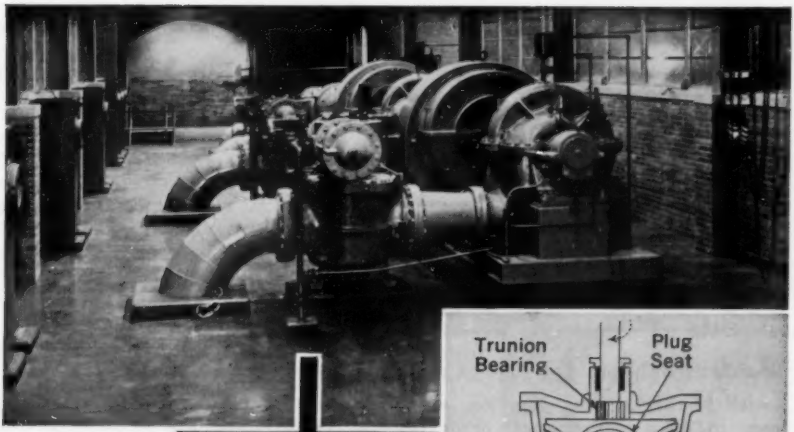

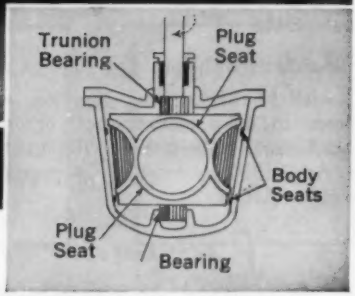

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Correspondence

Incalculable

To the Editor:

I wish to call to your attention what appears to be a mathematical error in K. L. Kollar's article on population studies (April 1956 JOURNAL). On page 430, the integral:

$$\int_0^r Ae^{-bx}(2\pi x)\,dx$$

is said to be equal to:

$$2A\pi \left[\frac{e^{-bx}}{b^2}\left(\frac{1}{bx}\right)\right]_0^r$$

or, when $r = \infty$, to:

$$2A\pi\left[1 - e^{-br}\left(\frac{1}{br}\right)\right]$$

A check of the calculus seems to indicate that the term in parenthesis in the above two expressions is incorrect. Thus, $\frac{1}{bx}$ should be $1 + bx$; likewise $\frac{1}{br}$ should be $1 + br$.

WILLIAM B. SCHWORM
St. Louis, Mo.; May 15, 1956.

To the Editor:

I was pleased to see my article in the April issue, but I have a slightly sour note to inject. As a result of an oversight and having the paper retyped several times, a $\div$ sign was substituted for a $+$ sign in two of the expressions in the manuscript. These read $1 \div bx$ and $1 \div br$, instead of correctly $1 + bx$ and $1 + br$.

K. L. KOLLAR
Caracas, Venezuela; May 28, 1956

The above letters were received independently of each other. Reader Schworm and Author Kollar appear about equally quick on the mathematical draw, as the latter, being abroad, did not see proof of his paper before publication.—ED.

Watershed Protection

To the Editor:

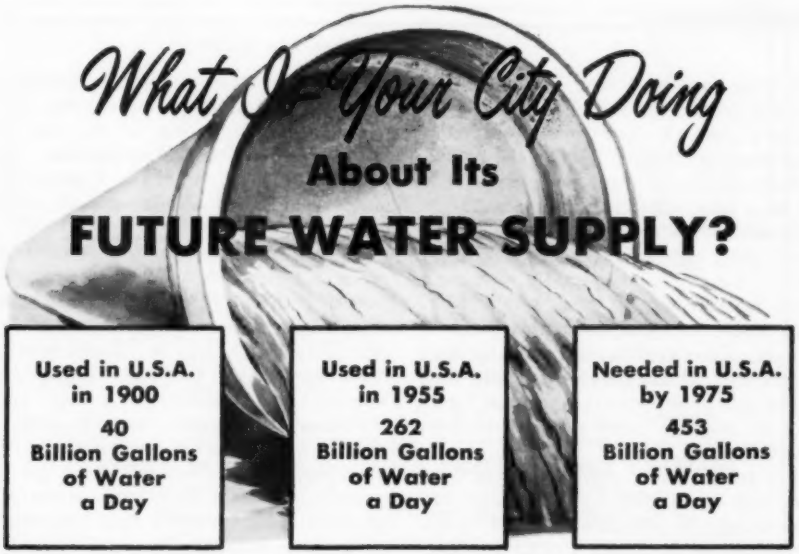
I do not consider myself competent to say yes or no to the recreational use of impounding reservoirs and their bordering lands as discussed by authors Dambach and Riehl (May 1956 JOURNAL), but I would like to make additional reference to two aspects of the problem:

1. Water is a primary product of the soil. Certain recreational uses, like public hunting and picnicking, have an adverse effect on soil conditions on a watershed. Wandering over the whole area, people do much damage by compacting the soil. When a soil is compacted, its total porosity is decreased. This results in reduced infiltration and percolation capacities of the soil and, therefore, in rapid surface runoff rates instead of a slow seepage flow. Furthermore, heavy recreational use of woodland is accountable in large part for decreased tree growth, as trampling compacts the soil and kills roots. These effects are often overlooked. Today water values transcend the value of all other land resources, including recreational use. It is my opinion that certain recreational uses like hunting, camping, and picnicking should be kept to a minimum because of their adverse effect on production of water by the soil.

2. I disagree with Mr. Dambach's statement that the practice of planting pines on the water's edge "has been overdone in many situations" and that the pines "are not the great conservers of water which so many people believe them to be." Nothing better protects the reser-

(*Continued on page 54 P&R*)

What Of Your City Doing About Its FUTURE WATER SUPPLY?



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(Continued from page 52 P&R)

voir from hardwood litter than a belt of conifers around the shoreline. As to the author's remark that pines create a "nightmare of tangled lines to the person attempting to cast from shore," would it not be a good solution to leave a 25- or 30-ft wide strip along the shoreline free from any riparian vegetation except grass? This certainly would give enough space to a fisherman, and at the same time would increase the water yield in the lake. The riparian vegetation, through its access to a constantly saturated soil, causes much heavier transpiration losses than the average forest type in our zone of climate.

The annual water requirements of trees vary with the species. The minimum annual amount of precipitation used by the various types of forest species is somewhat as follows: more xerophytic conifers (pines), about 3 in.; more xerophytic hardwoods (oaks), about 5 in.; more mesophytic conifers (spruces), about 8.5 in.; and more mesophytic hardwoods (beeches), about 10 in. (see US Dept. of Agriculture Miscellaneous Publication No. 257, p. 81). Accordingly, pines are greater conservers of water than hardwoods.

To Mr. Riehl's statement that evergreen plantings "provide added protection to both the reservoir and the water supply in various ways," I would like to add that a mixed pine-hardwood stand is more beneficial to the water supply than a pure pine stand. In mixed pine-hardwood stands, the litter decomposes rapidly and is easily incorporated into the mineral soil. Such soils have greater porosity and can store more water and transmit it more rapidly than soils under pure, dense pine stands.

WALTER C. SUSHKO

Baltimore, Md.
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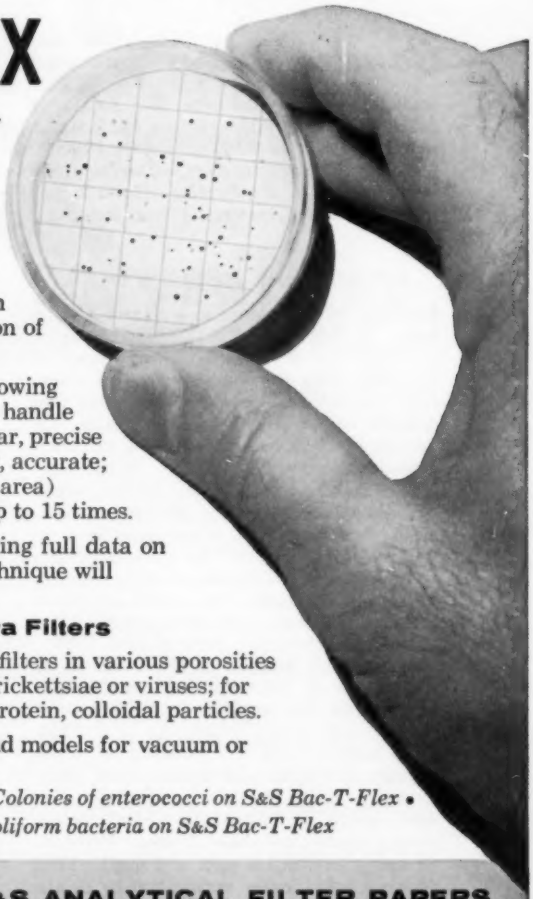

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Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

DAMS AND RESERVOIRS

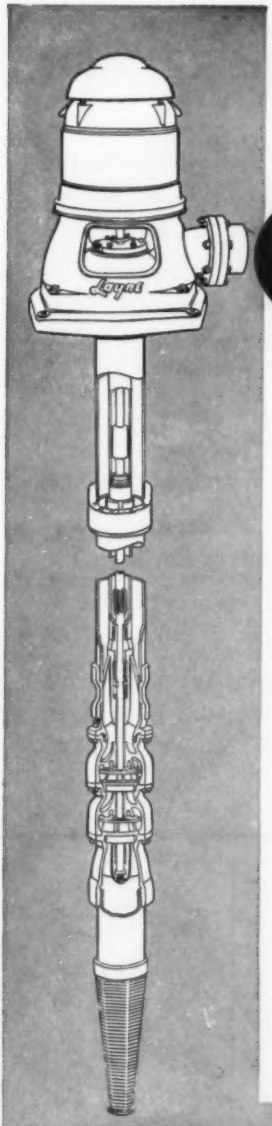
Behavior of Petroleum in a Water Reservoir. A. I. Iz'YUROVA. Gigiena i Sanit. (USSR), No. 5:15 ('55). Observations on petroleum films on water surface in reservoirs showed that rate of spreading of film depends on surface tension of water, i.e. thickness of film may vary from 2 to 10 mμ depending on purity of water. While low-boiling fractions evap. fairly rapidly, entire film disappears in time by settling and bacterial oxidation, rate being dependent on meteorological factors. In quiescent reservoirs, settled petroleum mixes with sediment and forms alternating layers with silt corresponding to summer and winter seasons.—*CA*

Experimental Studies of the Problem of Regulation of Chlorobenzene in Community Water Reservoirs. P. F. OBUKHOV. Gigiena i Sanit. (USSR), No. 7:7 ('55). PhCl does not affect O content in water storage reservoirs and shows very slight hindering action on bact. growth for few days, which is replaced by slightly accelerating action after some 10 days. Permissible concn. of PhCl is estd. at 1 mg/l. PhCl does not affect water purification by chlorination; this process somewhat reduces concn. of PhCl in water. Rabbits fed 0.1–1.0 mg/g PhCl over 14 months showed no deleterious wt. changes, had normal feeding habits, and showed no abnormalities in circulatory system in respect to blood compn. However, animals on highest dose level showed numerous structural disturbances in all internal organs; at 0.1 dosage, hepatitic and renal changes could be detected. Apparently this is threshold toxic dosage.—*CA*

The Condition of Water in Impounding Reservoirs for Water Supply. J. MULLER. Wasserw.-WassTechn. (Ger.), 3:18 ('53); GesundhSng., 75:168 ('54). Author discusses importance of limnological methods of investigating processes taking place in impounding reservoirs. Various sections deal with limnology as research problem; the formation of reservoirs and natural changes; temperature, thermal stratification, and currents; general chemistry; distribution of plankton and sludge; chemical and biological stratification; plankton and problem of taste and odor. Conditions in Klingenberg reservoir are described. Main problems discussed are formation of tastes and odors and effect of retention in reservoir on pathogenic bacteria. With sufficiently long period of retention destruction of pathogenic bacteria, especially typhoid bacteria, is possible but temporary reduction in period at times of high flow make chlorination, as well as filtration, necessary for reservoir water. Formation of tastes and odor depends on incoming water and also on changes taking place in reservoir. Great importance is attached to construction and careful supervision of sedimentation basins ahead of the reservoirs to reduce the amount of suspended solids. Causes of tastes and odor include increase in content of iron and manganese, humus, living or dead plankton, and flooding of uncleared land. Problem of cleaning out reservoirs used for supply requires investigation and collection of existing information on conditions in impounding reservoirs should be made.—*WPA*

The Fundamental Features of the Hydrochemical Regime and Microbiological Processes in the Rybinsk Reservoir. N. A. MOSEVICH & M. V. MOSEVICH. Trudy Problem. i Temat. Soveshchaniĭ, No. 2, Problemy Gidrobiol. Vnutren. Vod, No. 2:11 (USSR) ('54). Report of investigations during 1946 and 1948 on the compn. of waters of rivers and lakes that feed Rybinsk reservoir and of reservoir itself. Data cover seasonal variations of anions and cations in soln., Cl, SO_4^{--}, HCO_3^-, K, Na, Mg, and Ca, and quantity of org. matter and P in mud at bottom of reservoir.

(*Continued on page 66 P&R*)



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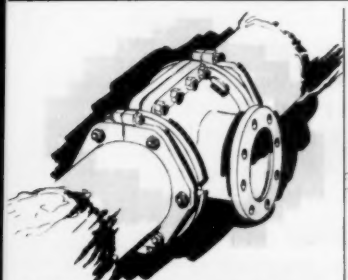
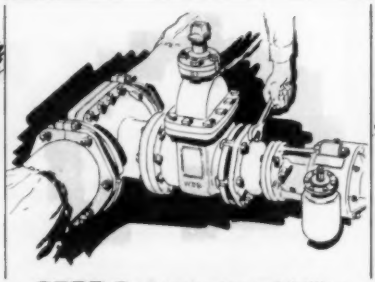
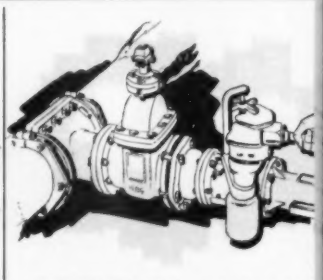


STEP 1: Attach sleeve. Clean main of rust and scale. Position and bolt tapping sleeve to main. Tighten mechanical joints or calk and lead sleeve.



STEP 2: Attach valve and drilling machine. Bolt tapping valve to tapping sleeve. Attach hand or power operated drilling machine to valve. Open valve wide.



STEP 3: Make cut. Move bori bar forward until it strikes main. Retract t turns–engage automatic feed–make cut. Wh cut is completed, retract cutter which retai coupon. Close valve–remove drilling machi

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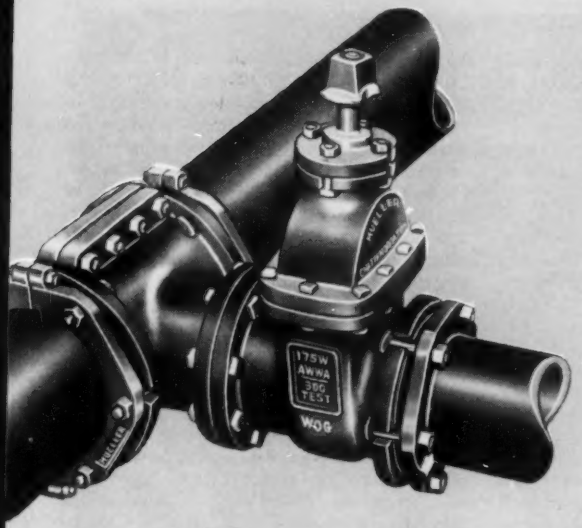
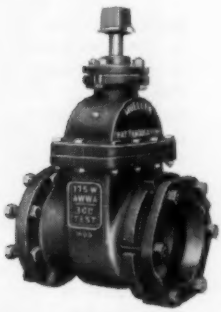


Tapping Cross. Calked or mechanical joint type. Nominal size fits all classes of cast iron pipe.



Tapping Valve. Calked or mechanical joint type. Perimeter pressure seating of discs. Conventional packing or "O" ring stem seal.

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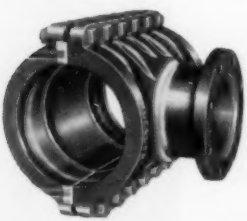
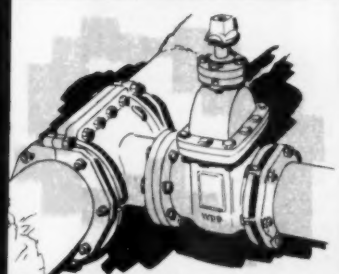
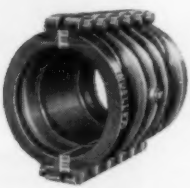


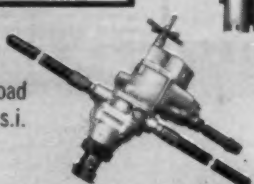
Tapping Sleeve. Calked or mechanical joint type. Nominal size fits all classes of cast iron pipe.



STEP 4: Make connection. Slip branch main into valve and tighten mechanical joint or calk and lead main to valve. Open valve and branch main is activated.



Split Repair Sleeve. Calked or mechanical joint type. One side of sleeve has tapped outlet with plug for escape of fluid while the sleeve is tightened around the leak.



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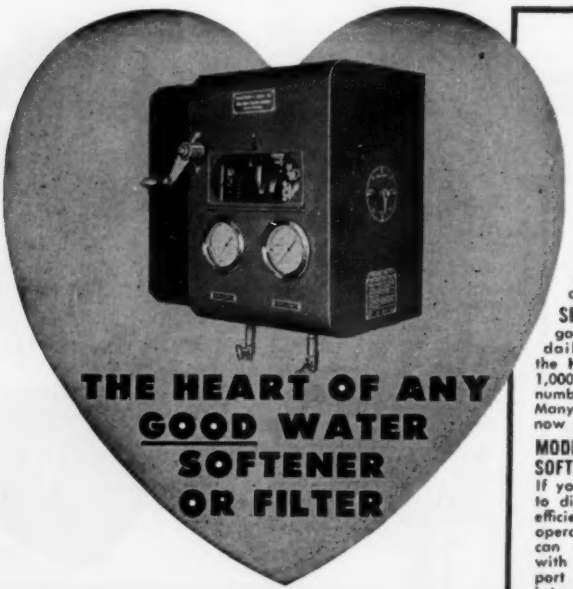
(*Continued from page 62 P&R*)

At same time, sol. O_2 and CO_2 were detd. Higher P content in reservoir waters is ascribed to P in soil of areas flooded. Total quantity of minerals is generally lower in reservoir than in waters of rivers that feed it. This is explained by inflow of flood waters during rainy season.—*CA*

Basic Features of Biogenous Compounds in the Reservoirs of the Volga System, in Connection With Their Primary Production. P. P. VORONKOV. Trudy Gosudarst. Gidrolog. Inst. 1953, No. 37:62; Referat. Zhur., Khim., No. 8376 (USSR), ('53). To establish potential productiveness of Kuibyshev and Stalingrad reservoirs, study was made of biogenous compds. of Lower Volga and Rybinsk reservoir. Water of Volga in region of Astrakhan contained 0.030–0.040 mg phosphates/l in summer and 0.060–0.070 in flood period; in summer it contained 0.025–0.050 mg nitrates/l, in winter 0.010 mg. In Rybinsk reservoir there were 0.100 mg phosphates/l in autumn, in winter 0.030, in flood period 0.150, and in summer 0.015 mg; content of nitrates in autumn-winter period was > 0.100 mg N/l; during spring floods their amt. decreased, and in June reached zero. Seasonal changes in nitrites and Si compds. had no simple connection with development of phytoplankton. Content of nitrites—0.000–0.020 mg N/l to end of winter—was at min. during summer, and during floods it increased. Si was at max. during winter (1.400 mg/l); during growing season there was 0.400–1.000 mg Fe/l, and during autumn and winter it was 0.025 mg. During floods, Fe was 0.070 mg/l, but during summer it decreased sharply, similarly to nitrates and phosphates. Variation in content of phosphates and Fe during one day illustrates activity of phytoplankton. During period of increased development of phytoplankton, daily consumption of phosphates for photosynthesis layer (0–2 m) comprised 0.045 mg/l. Development rate of phytoplankton was detd. by chlorophyll concn. in acetone exts.; these concns. varied in terms of pigment from 14 to 193 in 1946 and from 17 to 440 in 1947.

(*Continued on page 68 P&R*)



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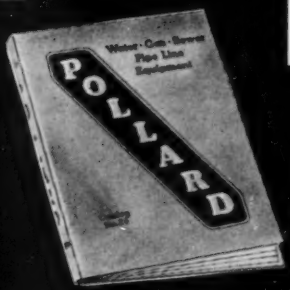

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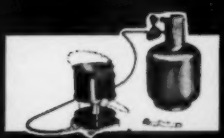

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(*Continued from page 66 P&R*)

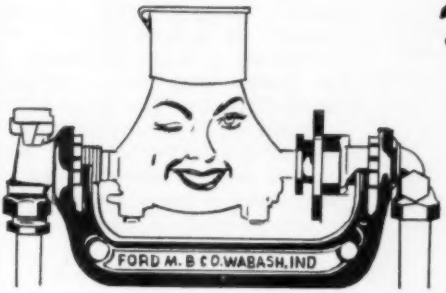
On basis of daily observations in calm weather, factor limiting development of phytoplankton proved to be N compds. Judging by consumption of phosphates, change in O_2 satn., and change in pH, development of algae continues after nitrates are gone, although chlorophyll content barely changes. Thus many small algae, which disappeared at night, existed in 0.5-m layer. At night, algae died out (about 50% of their amt.), and they dropped down to bottom at rate of 2 m/hr. Max. daily production of algae in the Rybinsk reservoir, on basis of O and consumption of phosphate, equaled 90 mg/l on avg; mean daily production was about 22 mg/l, that is, 178,640 tons for 0–2 m layer of water in reservoir. After 90 days of growing season, this comprised 16,000,000 tons. —*CA*

POLLUTION CONTROL

Experience With Solid Platinum Electrodes in the Determination of Dissolved Oxygen. I. University of North Carolina Studies. W. R. Lynn & D. A. Okun. Sewage and Ind. Wastes, 27:4 ('55). Use of rotating Pt electrode in place of dropping Hg electrode is suggested for detn. of DO, particularly for sewage and activated sludges. However, need for addn. of salt as electrolyte and filming of electrode by sewage precludes its use for this detn. **II. Georgia Institute of Technology Studies.** R. S. Ingols. Ibid: 7. Use of stationary Pt electrode, calomel half-cell electrode, and alternating charge works well for DO detns. where there are only small amts. of biol. breakdown products present, but will not work for detn. of DO in sewage. Coating Pt electrode did not get around this difficulty. Rotating Pt electrode works very well for detg. DO in waters which are free of floc and which are highly saline; therefore, its use for DO detns. in sewage and cultivated sludges is not recommended.—*CA*

Report Upon the Work of the Department of the Government Chemist for the Year Ending 31st March, 1954. H. M. Stationery Office, London, 30 pp. ('54). In-

(*Continued on page 70 P&R*)



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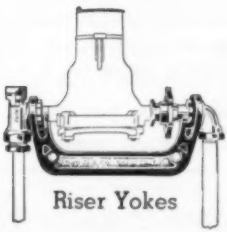

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(Continued from page 68 P&R)

formation given in Water and Sewage Div. of Govt. Chemist's annual report (pp. 12–17), includes report on examn. of water samples to investigate cause of pollution of domestic water supplies, death of fish in fountains in Kensington Gardens, causes of corrosion, and suitability of certain water supplies for boiler feed water. Surveys of rivers Usk and Tyne were continued. Paper has been prepared by Div. on methods for controlling addn. of fluoride to water, giving full details of dosing and method of anal. Other studies of fluoridation of drinking water included an investigation of availability and suitability of various fluoride chem. and examn. of urine from children aged 5–11 to assess amt. of fluoride assimilated from normal diet. Work has been undertaken on polluting effect on ground water of water percolating from gravel pits used as tips for household refuse. Lab. expts. have shown that effluent will have chem. and bact. polluting effect, and further expts. are planned to det. amt. of purification which takes place during passage of effluent through different gravels as found in gravel pit. Investigation into problem of synthetic detergents in sewage and river water was commenced by studying anionic detergents, particularly of alkylaryl sulphonate type, and the method detn. devised by Degens was used. Difficulties were met with concerning apparent loss of detergent (6–20%) in this method and further study of problem is planned using radioactive tracer technique. Use of sodium lauryl sulphate as std. for expressing results of detergent detn. is being investigated.—*WPA*

Biochemical Oxidation Characteristics of Stream Pollutant Organic Chemicals. M. B. ETTINGER. Ind. Eng. Chem., **48**:256 ('56). Factors affecting biol. attack are discussed. Stream pollutants cannot be evaluated satisfactorily in terms of BOD alone. Chemical resistant to biol. destruction may be capable of doing much more damage than depletion of O_2 in stream.—*CA*

(Continued on page 72 P&R)

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GASKET SEALER COMPOUND
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(Continued from page 70 P&R)

SOURCE OF SUPPLY—QUALITY

Chemical Characteristics of Delaware River Water, Trenton, New Jersey, to Marcus Hook, Pennsylvania. C. N. DURFOR & W. B. KEIGHTON. USGS Water-Supply Paper No. 1262, 173 pp. ('54). Delaware R. is principal source of water for many industries and municipal water supplies along this reach, and it is used for industrial and municipal waste disposal. 8 observation stations were spaced along stream section, and hydrographic and anal. data were accumulated over 40-mo period. Water anal. (ppm): SO_4^{--}, 12–382; Cl^-, 10–2,020; F^-, 0.9; NO_3^-, 0.1–19; HCO_3^-, 11–71; Ca 7.9–75; Mg, 26–147; Na, 0.07–1,080. High values for Cl, Mg, and Na are due to sea water intrusion, which is noted as far upstream as Philadelphia during low-flow rates. Generally, mineral content and BOD increase from Trenton downstream, whereas DO decreases. Signif. O deficiency occurred in many samplings during late summer and fall months, especially in lower portion of river section. Anals. were made by std. tests and generally show appreciable increases in all contaminants proceeding downstream from station to station. Measurement of elec. cond. is proposed as convenient and rapid method for detecting sea water intrusions or presence of slugs of poln. —*CA*

Gulf Coast Inlet Converted to Fresh-Water Reservoir. C. A. WEISE. Civil Eng., **24**:60 (Oct. '54). Turtle Bay project is one of first major attempts to convert natural bay or inlet along Gulf Coast into reservoir for fresh-water storage. Improvement and enlargement of Turtle Bay, once part of tidal Trinity Bay, provides irrigation water for about 125 rice farmers. New and reconstructed levees and spillway, el. 5.0, forms Lake Anahuac of 35,000-acre-ft capac. Source of water is runoff from Turtle Bayou watershed (130 sq mi) plus water pumped at Turtle Bay Pump Station from Trinity R. during adequate river flow. Anahuac Station, enlarged to 240,000 gpm has max.

(Continued on page 74 P&R)

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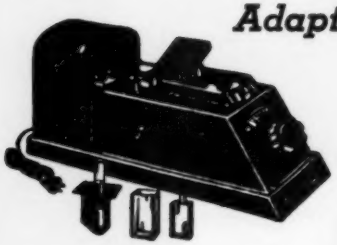

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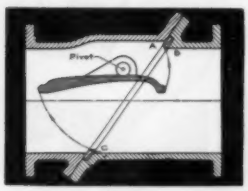

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(Continued from page 72 P&R)

lift of 25′ and can pump water to irrigation canal either from reservoir or Trinity R. Project will provide water for 25,000 acres plus anticipated industrial use. Chambers-Liberty Counties Navigation Dist. financed project by means of $1,250,000 bond issue payable through revenues.—*PHEA*

Sewage Effluent Reclamation for Industrial and Agricultural Use. A. E. Greenberg & J. F. Thomas. Sew. Ind. Wastes, **26**: 761 ('54). Study of water reclamation by spreading of sewage treatment plant effluent (10 ppm BOD) and settled sewage (100 ppm BOD) in Hanford fine sandy loam gave bacteriologically safe water at percolation rate of ½ acre-ft/day/acre/when the sewage passed through at least 4′ of soil. There was little chem. change in reclaimed water. Need for insect and algae control are to be expected.—*CA*

Quality of Surface Waters of the United States, 1951, VII–VIII. Lower Mississippi River Basin and Western Gulf of Mexico Basins. S. K. Love. USGS Water-Supply Paper No. 1199 ('55). Tabulated data are given from more than 100 river collection stations situated in 10 southwestern states. Chem. analysis includes silica, Fe, Ca, Mg, Na, K, HCO_3^-, SO_4^{--}, Cl^-, NO_3^-, and in many instances K, F, and B. Also included in most instances are data on suspended sediment, temp., pH, specific conductance, and color.—*CA*

Quality of Surface Waters for Irrigation, Western United States, 1952. S. K. Love. USGS Water-Supply Paper No. 1362 ('55). Chem. anal., chiefly salt burden, and other phys. measurements are given for network of observation stations west of Mississippi R. Highest in DO was the Pecos R. (Orla, Tex.) with 9.22 tons per acre-ft of runoff. Other rivers having high salt burdens (greater than 1.0 ton per acre ft) were Powder, Cheyenne, South Platte, Saline, Arkansas, Canadian, Red, Washita, Gila, and Sevier. Low burdens (less than 0.1 ton per acre-ft) were found in Mokelumne,

(Continued on page 76 P&R)

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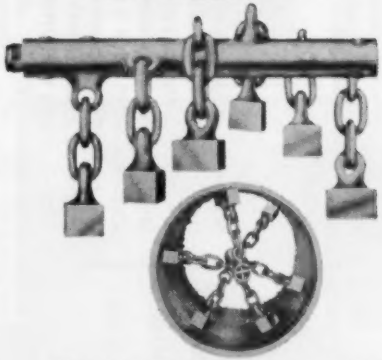

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(*Continued from page 74 P&R*)

Feather, American, and Willamette. Consistently high Na-adsorption ratios, $Na^{+}-[Ca^{++}+Mg^{++}/2]^{\frac{1}{2}}$, were found in Cimmaron, Canadian, San Jacinto, lower Rio Grande, Pecos, and Gila.—*CA*

SWIMMING POOLS

Chaetopoda in Swimming Baths and Filter Plants. GUTSCHE. Arch. Bäderwesen, **6**:214 ('53). Chaetopod worms introduced by town water supply into swimming bath at Oldenburg were destroyed in bath by addition of 11 mg/l of Na hypochlorite. In sand filter they were not destroyed when filter was allowed to stand for ¾ hr with 7 l of hypochlorite solution (in 9 cu m) or for 1½ hr with 10 l. Desired result was only achieved by recirculating 30 cu m of water containing 7 l of Lysolin for 4 hr. Green color increased in water after treatment and was only removed after re-maturing of filter. —*WPA*

Effective Algae Control. R. C. SHERWOOD. Beach and Pool, **29**:25 ('55). Author states that use of potent algicide added to pool water at specific periods and proper amts. will prove successful in preventing algae growths. The use of algicide for this purpose, however, does not replace but supplements use of Cl. Alkyl-dimethyl-benzyl-ammonium chloride is presented as efficient sanitizing agent and algicide. Best results are obtained if pool is cleaned and filled with fresh water before algicide is first used. Specific directions for effective use of this algicide are discussed.—*C. C. Johnson, Jr.*

Note on the Chlorination of Water in the Presence of Traces of Natural Bromide. J. K. JOHANNESSON. New Zealand J. Sci. Technol., **35**:600 ('55). Presence of bromide interferes with process of chlorination of natural waters. This effect appears to be commonly overlooked in the literature. Monobromamine is shown to have an absorption max. in the ultra-violet of 277 mμ. Studies initiated in this lab. reveal that bromide is often present in significant concentration and that in its presence, anomalous

(*Continued on page 78 P&R*)

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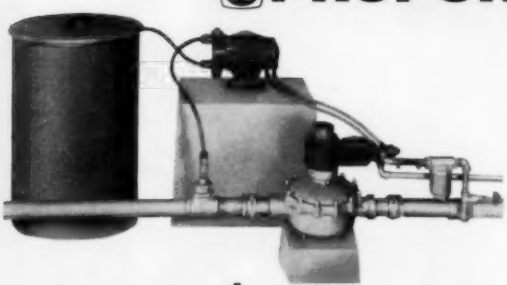

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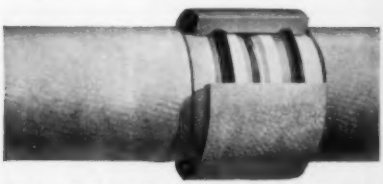

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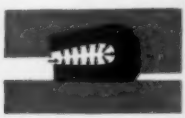

Water pressure expands rings; the higher the pressure, the tighter the seal.

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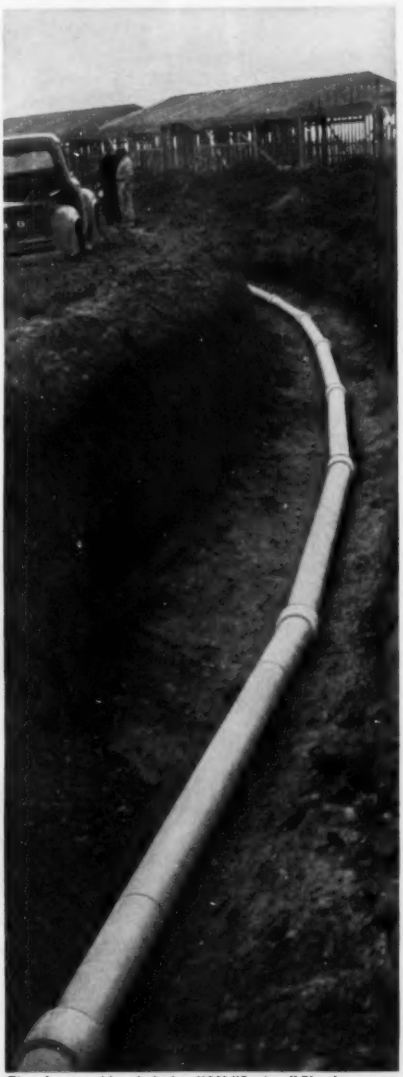

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(*Continued from page 76 P&R*)

behavior of chlorination is to be expected. Detailed investigation of effect of bromide upon breakpoint process is necessary for proper control of sea-water pools.—*PHEA*

Engineering a Swimming Pool. H. FEITLER. Public Works, **86**:145 ('55). Details of present-day swimming pool design are given. Reference given to APHA pamphlet on recommended practices and Ill. State Requirements. Assumed pool of 278,500 gal. and turnover of every 6 hr, required 775 gpm. Drains should be sized for even more rapid turnover and grate opening should be 10 times outlet pipe area. Inlet piping should not permit any dead spots in pool and should proportion incoming water on basis of vol. of water at incoming point. Valved inlets permit regulation and reduce entrance velocs. Inlets should be 24" above floor in deep portion and as near floor as possible in shallow section. Scum gutter overflow drains should be on 15' centers all around pool and 2½" pipe size as a min. All gutter water should be salvaged. Detailed pump operated system is described with pair of electrodes, and relay and normally closed solenoid valve are used. Deck drains should not be connected to gutter drains but discharge to waste. Vacuum cleaner fittings 2" size will require a min. of 3" header. C.i. pipe used if a pipe tunnel is used. Diatomite filters should permit water to be of less than 1 ppm turbidity. Program outlined includes 400-gal surge and inlet tank pool drain, vacuum connection, 10" hair and lint trap—15 HP. motor driven pump, 200-gal slurry feeder, 258-sq ft filter, pool heater, 35 pound chlorinator, 10" flow meter and jet precoat pump. Makeup water should be filtered before entering pool. A room 30' × 12' plan and 8' high will suffice. No alum or soda ash is used. About 28 pounds diatomite is used daily for 278,500-gal pool. No special abilities required for operators of such layout. About 1 hr per day is required for equipment, maintenance and operation. Valves should be readily accessible and pipe lines oversized for more stringent requirements that will come. Design can insure success of pool.—*E. L. Filby*

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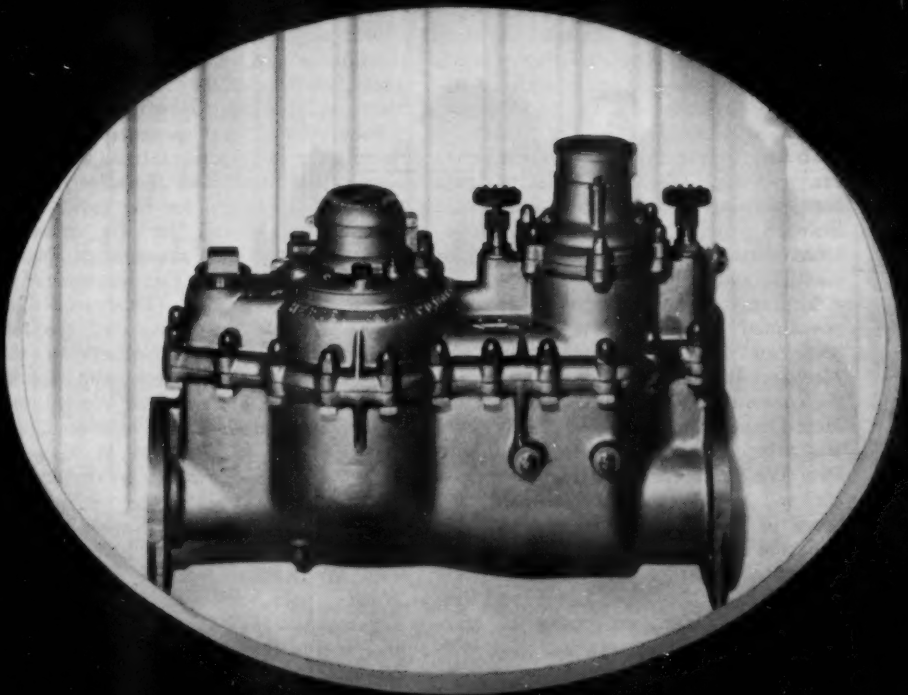

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Section Meetings

Arizona Section: The Arizona Section and the Arizona Sewage & Water Works Assn. held their annual meetings jointly, at the Buena Vista Hotel, Safford, Apr. 5–7, 1956. The total registration was 164. The meeting opened Thursday morning, Apr. 5, with an address of welcome by O. O. Rawson, mayor of Safford. G. Ted Rekerdre, chairman of the Arizona Section, gave the response. Technical sessions, held both morning and afternoon every day, included papers on pipe installation, instrumentation, water use, corrosion, chlorination, and training.

Thursday afternoon a very festive "get acquainted" reception was held at the home of Margaret Frederickson and Margaret Rhoads for the ladies attending the convention. Thursday evening a No Host Dinner was held. On Friday there was a luncheon at the Safford Inn, with Paul Weir as speaker. Friday afternoon Mr. and Mrs. A. Frederickson held open house in Thatcher.

At the annual banquet Friday evening, Chairman Rekerdre presided, with Byrl D. Phelps, of Municipal Engineers, Inc., San Diego, Calif., as master of ceremonies. An Indian theme prevailed, with the ladies in the traditional squaw dresses and the tables beautifully decorated in suitable fashion. Principal speaker was H. B. Rasch, Infilco Inc.

At the business luncheon meeting on Saturday, the following officers were elected: chairman—M. V. Ellis, supervisor, Sewage Treatment Plant, Div. of Water & Sewers, Phoenix; vice-chairman—Quentin M. Mees, associate professor of civil engineering, University of Arizona, Tucson; secretary—H. C. Biggle-stone, Board of Fire Underwriters of the Pacific, Phoenix. John Francis Rauscher, hydraulic engineer, Tucson, was chosen to receive the Fuller Award. The meeting closed with a most colorful Hawaiian *luau* at the Safford Elks Club Saturday evening, following a most enjoyable cocktail hour sponsored by all the manufacturers' representatives attending. Exotic flowers used in the table decorations and as favors for the ladies were flown from Hawaii for the occasion. Clark Webb, district manager of Pacific States Cast Iron Pipe Co., was master of ceremonies. A. W. (Dusty) Miller, Johns-Manville Corp., installed the following newly elected members of the Sludge Shoveler's Society with appropriate rites: P. F. Pew, Paul Weir, Louis R. Ingram, and D. S. McFarland.

QUENTIN M. MEES
Secretary-Treasurer

Kansas Section: The Kansas Section held its eleventh annual joint meeting with the Kansas Sewage & Industrial Wastes Assn. at the Jayhawk Hotel, Topeka, Apr. 4–6, 1956. The attendance of 236 was the largest, thus far, in the existence of the two groups. Separate sessions were held Wednesday (Apr. 4) afternoon and Friday morning, and a general meeting was held all day Thursday. Technical papers presented covered such topics as hydraulic measurements, pipe specifications, chlorinating equipment, taste and odor control, maintenance of fire hydrants, highway legislation, industrial-water reuse, pumps, radioactivity, weather control, rates, water shortages,

(*Continued on page 84 P&R*)

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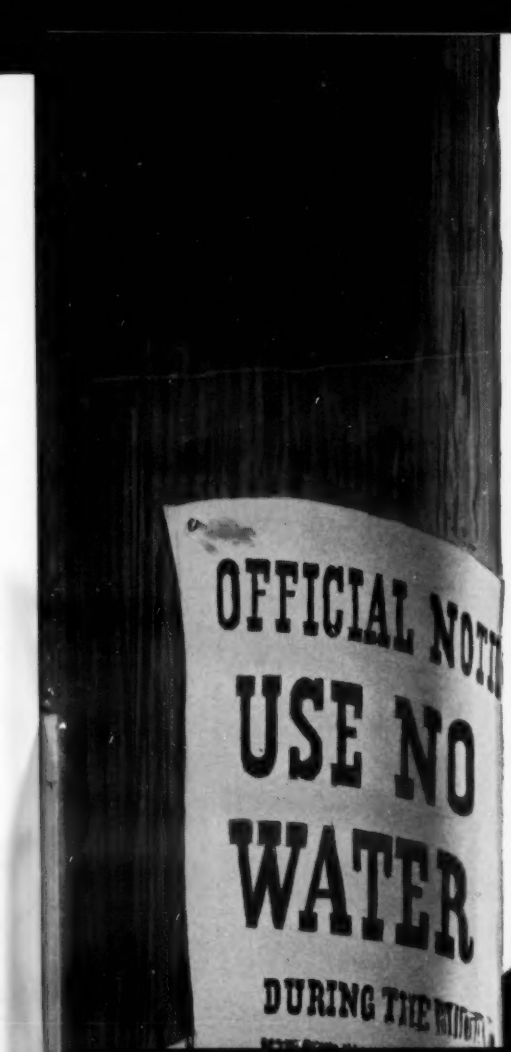

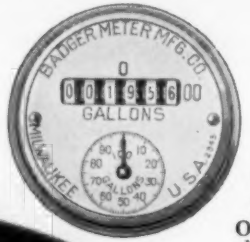

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Section Meetings

(Continued from page 80 P&R)

cathodic protection, meter testing and maintenance, and well construction.

At the buffet supper Wednesday evening, Jack Moranville, Mississippi Lime Co., acted as an eminently capable MC for Fred Townsend's (Ash Grove Lime & Portland Cement Co.) highly entertaining amateur floor show, and also gave an accomplished performance himself. Thursday evening more than 200 were in attendance at the annual banquet held at the Roof Garden of the Jayhawk Hotel. Guy Keeler of the University of Kansas gave the address. Twenty-six water works operators' certificates were presented by Howard A. Stoltenberg, secretary of the Certification Examining Board. Orville Kuran received the Section's Past Chairman's Award, Dorr Pelton doing the honors. Albert W. Rumsey, chief chemist, Board of Public Utilities, Kansas City, was nominated to receive the Fuller Award, Robert Hess, chairman of the committee, making the announcement and presentation of the committee's signed letter with citation.

The following officers were elected Friday at the business meeting held immediately after the annual closing luncheon: B. H. Vanblarcum, chairman; Roy F. Bluejacket, vice-chairman; Robert J. Schenk, trustee; and H. W. Badley, secretary-treasurer.

H. W. BADLEY
Secretary-Treasurer

Nebraska Section: Lincoln was again the scene of the annual section convention Apr. 11–13, conducted jointly with the Utilities Section of the League of Nebraska Municipalities. The very fine meeting included a full schedule of excellent technical papers and enjoyable social events. (A list of water works papers and their authors will be published in the December 1956 JOURNAL.) The annual

(Continued on page 86 P&R)

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Fact #3 **ECONOMY** – First cost of Alco electric welded steel pipe is low. Installation cost reduced by long lengths, light weight. More strength against bursting per dollar than other type of transmission pipe.

Fact #4 **EFFICIENCY** – Smooth, high-gloss tar-enamel inside surface gives maximum rate of flow, minimum friction loss.

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Fact #6 **MANY JOINING METHODS** – Alco electric welded steel pipe joints may be welded, flanged or Dresser-coupled. Can be joined to be leakproof under pressure conditions ranging from vacuum to thousands of pounds per square inch of water, air, gas or steam.

Fact #7 **WIDE SELECTION** – Alco electric welded steel pipe sizes, wall thickness and section lengths give you, in effect, a custom-tailored pipe. Special sections are available.

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New York

Section Meetings

(Continued from page 84 P&R)

banquet was one of the highlights of the meeting. The coveted Fuller Award nomination went to Bert Gurney, who has done a very great deal in the development of the Nebraska Section and has displayed a keen interest in water works affairs. Bill Orwig, director of athletics at the University of Nebraska, gave a humorous and informative talk on that subject. J. W. Van De Water, of the North Western Bell Telephone Co. in Omaha, showed colored slides of a 163-mile 10-day canoe trip he and his wife made down the Colorado River. Mr. Van De Water narrated the presentation and all shared in his very interesting experience.

At the business meeting the following officers were elected: Carl L. Fisher, chairman; George H. Beard, vice-chairman; John E. Olsson, secretary-treasurer; and Bert Gurney, director. Retiring Chairman E. Bruce Meier automatically became a member of the board of trustees.

Mrs. John W. Cramer provided a ladies program for the twelve Section wives in attendance.

JOHN E. OLSSON
Secretary-Treasurer

New York Section: The New York Section held its annual spring meeting at the Hotel Utica, Utica, on Apr. 18–20, 1956. The total registration was 309. The technical program [which will be fully listed in the December 1956 JOURNAL] covered various aspects of metering, cast-iron pipe inspection and purchasing, a description of the new Utica Board of Water Supply service building, distribution storage, training and certification, and main disinfection.

John T. McKennan, mayor of Utica, gave the address of welcome at the Thursday (Apr. 19) luncheon. Chairman Everett Rowley announced the election of John G. Copley, general manager, Elmira Water Board, as director and Carl W. Kidd, superintendent, Water & Sewer Dept., Danville, as trustee.

Prior to the banquet Thursday evening, a Club Room party was given through the courtesy of the Water & Sewage Works Manufacturers Assn. At the banquet Life Membership was bestowed on George H. Buck, Christopher T. Chenery, Grover R. Reckard and Maurice R. Scharff. Entertainment was provided by "The Colgate 13," who sang both new and old songs during and after the banquet. A beer party followed the banquet with Leo Aiello and his barbershop chorus entertaining.

KIMBALL BLANCHARD
Secretary-Treasurer

Pennsylvania Section: The eighth annual meeting of the Pennsylvania Section was held in Philadelphia, at the Bellevue-Stratford Hotel, Apr. 3–5, 1956. This was the most successful meeting of the Section, with a total official registration of 295. The program presented was well received and included 2 days of technical sessions. The subjects discussed included: use of the network analyzer (the device was demonstrated at City Hall Annex throughout the meeting), automatic coagulation control, training, hurricane problems, service line friction losses, future planning, extension financing, and treatment plant facilities.

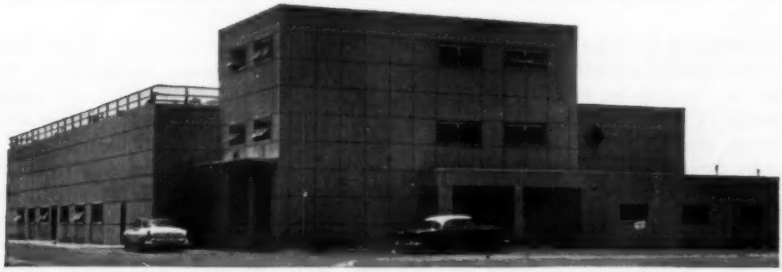
After the Wednesday (Apr. 4) luncheon, at which greetings were extended by S. S. Baxter, water commissioner of Philadelphia, and at which the guest speaker was Dr. James Creese, president, Drexel Institute of Technology, Philadelphia, an inspection tour of the Philadelphia water works was made by the men, while the ladies attended a bridge and canasta party.

At the business session Thursday afternoon the following officers were selected for the ensuing year: chairman—B. French Johnson; vice-chairman—P. W. Miller; secretary-treasurer—L. S. Morgan; and trustee (3 years)—Homer E. Beckwith.

The annual banquet was held Thursday evening with guest speaker Frank C. Amsbary Jr. giving news of the Association. The nominee for the Fuller Award

(Continued on page 88 P&R)

At St. Petersburg's new Cosme filter plant



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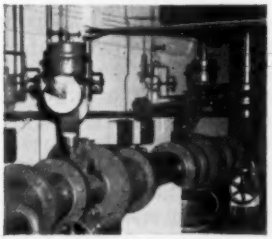
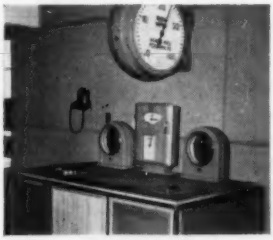


Filter gallery: Diaphragm pots for Master Pneumatic System are just above counterbalanced arms of the Simplex Controllers.



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Section Meetings

(Continued from page 86 P&R)

was announced as John H. Murdoch Jr. Life Membership awards were also presented to Francis S. Friel and Willard N. Greer.

Among the activities planned and enjoyed by those attending the meeting were the preconvention get-together Tuesday night and the cocktail hours Wednesday and Thursday, all sponsored by the Water & Sewage Works Manufacturers Assn.; the ladies toured historic sites in Philadelphia Thursday morning and then attended luncheon at the Warwick Hotel as guests of Mr. and Mrs. Friel; the men interested engaged in a golf tournament; and the coffee hours (new this year for the Section) were well attended by both sexes. It was felt that this was the best meeting in the 8-year history of the Pennsylvania Section.

L. S. MORGAN
Secretary-Treasurer

Southeastern Section: The Southeastern Section held its 27th annual meeting at the Bon Air Hotel, Augusta, Ga., Mar. 18–21, 1956, with 319 members and guests registered. The meeting was formally opened Monday morning (Mar. 19) with a brief but hearty welcome by Hugh L. Hamilton, mayor of Augusta. Monday afternoon was devoted to an inspection of the very modern power generating plant of the South Carolina Electric & Gas Co. near Augusta, with many also visiting the modern and well operated water purification plant of the city of Augusta.

The technical program dealt with weather modification, distribution systems, water district development, tastes and odors, industrial-water demand, safety practices, and casualty insurance.

At the business session Tuesday noon Carl C. Lanford, engineer and superintendent, Commissioners of Public Works, Greer, S.C., was elected chairman; L. E. Wallis, superintendent, Water & Light Dept., Blakely, Ga., was elected vice-chairman; and Lewis R. Simonton, superintendent of water purification, Griffin, Ga., was elected trustee. The Executive Committee had previously reelected N. M. deJarnette secretary-treasurer. Other business included adoption of a resolution urging members of Congress from Georgia and South Carolina to support legislation relieving water utilities of the cost of relocating their facilities.

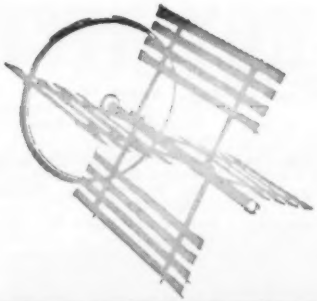
Social activities included a "Get Acquainted Party" Sunday evening; a barbecue, bingo game, and club room entertainment Monday evening, and a cocktail hour, banquet, and dance Tuesday evening. Part of these activities were sponsored by the Water & Sewage Works Manufacturers Assn. and part by the city of Augusta. Ben A. Hinson (chairman), Denzle Q. Whitehair, and Charles F. Lynn, serving as the Club Room Committee for WSWMA, and Leopold Mothner and Marion B. Cooper, serving as the city's Local Arrangements Committee, had planned all details in a most thorough manner and received high praise and sincere thanks from the entire assembly.

Charles Floyd, Badger Meter Co., Greenville, S.C., entertained and thoroughly baffled the audience at the Monday evening party with a series of magic feats, and Joseph W. White, Pure Oil Co., Atlanta, speaking at the banquet, kept the guests in a hilarious mood with his witty remarks.

William R. Wise, manager and engineer of public works, Greenwood, S.C., who as chairman of the Program Committee had been largely responsible for the technical program, also served as toastmaster at the banquet, presenting a plaque and gavel to Retiring Chairman Bruce J. Sams, Savannah. Twenty-five-year membership pins were awarded to J. G. Beacham, Paul Killian, and N. M. deJarnette. The Fuller Award Committee announced that Robert C. Kauffman had been chosen as the Section's nominee.

The visiting ladies were delightfully entertained on Monday with a tour of Augusta's scenic and historic spots and on Tuesday with a bridge luncheon at the Augusta Country Club, these events having been arranged by Mrs. Julia Clark, chairman, and others on the Ladies Entertainment Committee.

N. M. DEJARNETTE
Secretary-Treasurer



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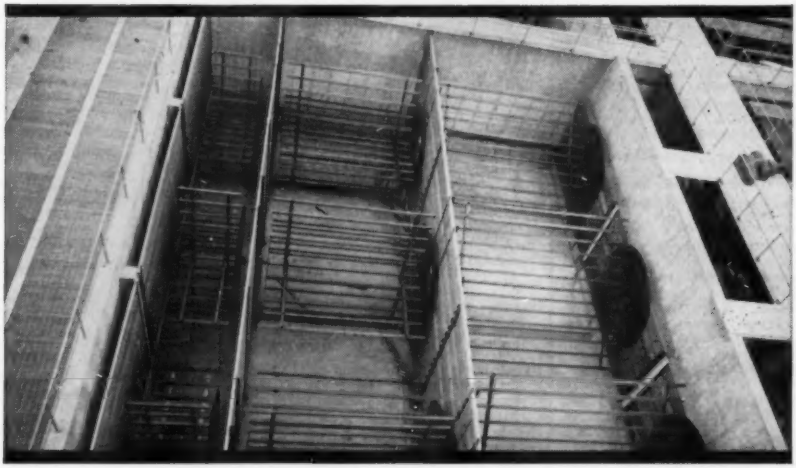

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NEW MEMBERS

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Abbott, Wells O., Jr., Civ. Eng. Asst., Dept. of Water & Power, 207 S. Broadway, Los Angeles 54, Calif. (Apr. '56) *D*

Ackermann, William C., Chief, State Water Survey, Box 232, Urbana, Ill. (Apr. '56) *R*

Ahrens, Alden Frederick, Chief Water Service Inspector, Div. of Water, 565 N. Erie St., Toledo, Ohio (Apr. '56) *D*

Akers, Ward L., San. Engr., State Dept. of Public Health, 121—4th Ave., Rock Island, Ill. (Apr. '56) *RP*

Alexander, Richard D., Engr., Baldwin & Cornelius Co., 117 W. Sunrise Hwy., Freeport, N.Y. (Apr. '56) *RPD*

Anderson, Marshall A., Vice-Pres.-Treas., American Water Works Co., Inc., c/o American Water Works Service Co., 3 Penn Center Plaza, Philadelphia 2, Pa. (Apr. '56) *M*

Barber, Arthur Houghton, Jr., Chief Engr., Walter Hook & Assocs. Inc., Box 1297, Charlotte, N.C. (Apr. '56) *P*

Barney, Robert A., Lab. Technician, Water Dept., City Hall, Oklahoma City, Okla. (Apr. '56) *P*

Bascom, W. Broughton, Pres., Sterling Water Co., 116 N. 5th St., Allentown, Pa. (Apr. '56) *M*

Becker, Dean C., Engr., St. Louis County Water Co., 8390 Delmar Blvd., St. Louis 24, Mo. (Apr. '56) *P*

Berkemeyer, Robert, Chemist, Grand River Dam Authority, Choteau Steam Plant, Pryor, Okla. (Apr. '56) *P*

Boehmer, August D., Salesman, Robert F. McGivern & Assocs., 1771 W. 5th Ave., Columbus 12, Ohio (Apr. '56) *PD*

Bowen, Marshall R.: *see* Central Basin Munic. Water Dist.

Bradley, Earl C., Jr.: *see* Tyler Pipe & Foundry Co.

Bradley, Milford E., Sales & Service, Warren Chem. Co., Inc., Chicago Heights, Ill. (Apr. '56) *MRPD*

Brighton, City of, Domenic Kiomento, Supt., Water & Sewerage Dept., 420 S. 3rd St., Brighton, Mich. (Corp. M. Apr. '56) *MRPD*

Brookes, Ned V., Director, Public Works, City Hall, Borger, Tex. (Apr. '56) *M*

Brown, George W., Jr., Asst. Chief Engr. & Supt., Water Dept., Kansas City, Mo. (Apr. '56) *MD*

Bryant, George T., Asst. Prof. of San. Eng., 508 Ames Hall, Johns Hopkins Univ., Baltimore 18, Md. (Apr. '56) *RP*

Burlingame, Roland S., Partner, Camp, Dresser & McKee, 6 Beacon St., Boston 8, Mass. (Apr. '56) *MPD*

Carlson, Richard W., Jr., Sales Repr., Neptune Meter Co., 1507 S. Grant St., Aberdeen, S.D. (Apr. '56) *M*

Cashman, Charles D., Engr., Clyde E. Williams & Assocs. Inc., 312 W. Colfax Ave., South Bend 1, Ind. (Apr. '56) *RPD*

Central Basin Munic. Water Dist., Marshall R. Bowen, Director, 7439 E. Florence Ave., Downey, Calif. (Corp. M. Apr. '56) *R*

Cheak, Alfred J., Secy.-Treas., Maywood Mutual Water Co., No. 1, 5953 Gifford Ave., Huntington Park, Calif. (Apr. '56) *MD*

Chelsea, Edwin D., Gen. Design Engr., Chicago Park Dist., Administration Bldg., 425 E. 14th Blvd., Chicago, Ill. (Apr. '56) *PD*

Chemical Testing Corp., Frederick M. Spitz, Gen. Mgr., 29–28—41st Ave., Long Island City 1, N.Y. (Corp. M. Apr. '56)

Ching, Theodore K. C., Civ. Engr., Board of Water Supply, Box 3410, Honolulu 1, Hawaii (Apr. '56) *PD*

Clabaugh, Ernest L., Administrative Asst. Plant Engr., Ravenna Arsenal Inc., Apco, Ohio (Apr. '56) *M*

Clark, Bill C., Supt., Galveston County Water Control & Irrigation Dist. No. 12, Kemah, Tex. (Apr. '56) *MPD*

Clayton County Water Authority, Robert E. McMullen, Mgr., Jonesboro, Ga. (Corp. M. Apr. '56) *MD*

Clough, Ross S., Water Supt., Village of Alsip, 4500 W. 123rd St., Blue Island, Ill. (Apr. '56) *MPD*

Coe, Harvey, Jr., Branch Mgr., Watertower Paint & Repair Co., Inc., Box 306, Clear Lake, Iowa (Apr. '56) *MP*

Columbia, J. Z., Mgr., Borough of Ambridge Water Authority, R.D. 2, Burgettstown, Pa. (Apr. '56)

Corlett, Thomas Bruce, Jr., Asst. Engr., Water Dept., Fridley Softening Plant, 43rd & Marshall Ave. N.E., Minneapolis, Minn. (Apr. '56) *M*

Courtney, Robert C., Asst. to City Engr., Denton, Tex. (Apr. '56) *MPD*

Crow, Harry B., Supt., Water & Sewer Utilities, 240—5th Ave. W., Kirkland, Wash. (Apr. '56) *MD*

Cunningham, Guilford, Supt., Water Works, Fort Pierre, S.D. (Apr. '56) *MPD*

Dague, Harold F., Vice-Pres., American Plastic Pipe Corp., 16616 Garfield, Fairmont, Calif. (Apr. '56) *MRPD*

Dalbke, Richard G., Head, Water Treatment Div., Tech. Sales Service, Dearborn Chem. Co., Merchandise Mart Plaza, Chicago 54, Ill. (Apr. '56) *P*

Day, Alfred V., Asst. Utilities Engr., State Public Utilities Com., State Bldg., San Francisco, Calif. (Apr. '56) *RPD*

Deacon, Romaine K., Sales Repr., Wallace & Tiernan Inc., 213 Carlton Terrace Bldg., Roanoke, Va. (Apr. '56) *RP*

DeBourge, Rudy B., Gen. Mgr., Commonfields of Cahokia Public Water Dist., 705 Upper Cahokia Rd., East St. Louis, Ill. (Apr. '56) *M*

De Falco, George, Asst. Civ. Engr., Design Div., Dept. of Water Supply, Gas & Elec., Municipal Bldg., New York 7, N.Y. (Apr. '56) *MRPD*

De Sola, Ricardo, Director of Aqueducts, Mijares a Mercedes No. 33, Caracas, Venezuela (Apr. '56)

Dettloff, A. F., Mgr., Rogers City Power Co., 221 N. 3rd St., Rogers City, Mich. (Apr. '56) *MD*

Dickerson, Bruce W., San. Engr., Hercules Powder Co., Delaware Trust Bldg., Wilmington, Del. (Apr. '56) *RP*

Dunn, Angus O., Supt., Water Dept., Staunton, Va. (Apr. '56) *MP*

Dusold, Edward H., Field Supervisor, Watertower Paint & Repair Co., Inc., Box 306, Clear Lake, Iowa (Apr. '56) *MP*

Dykhuis, Elmer P., Supervisor, Water & Sewage Works Sales, Crane Co., 836 S. Michigan Ave., Chicago, Ill. (Apr. '56)

Easterday, Robert Elton, Asst. Div. Engr., Water Dept., 1640 S. Kingshighway, St. Louis 10, Mo. (Apr. '56) *D*

Engeman, George H.; *see* Loudoun Water Corp.

Engine & Equipment Co., Emmett G. Simmons, 2102 E. 28th St., Long Beach 6, Calif. (Assoc. M. Apr. '56)

Epstein, Arthur J., Chemist, St. Louis County Water Co., 8390 Delmar Blvd., St. Louis 24, Mo. (Apr. '56) *P*

Fabing, Lester G., Supt., Water Works, Light & Water Dept., 119 W. Walnut Ave., Lompoc, Calif. (Apr. '56) *MRPD*

Fago, Anthony, Pres., Fago Brothers Contracting Co., Inc., 521 Young St., Tonawanda, N.Y. (Apr. '56) *D*

Fago, Joseph, Vice-Pres., Fago Brothers Contracting Co., Inc., 521 Young St., Tonawanda, N.Y. (Apr. '56) *D*

Fee, Arthur E., Sr. Engr., J. L. Richards & Assocs., 50 Glencairn Ave., Ottawa 1, Ont. (Apr. '56)

Feingold, Morton, Civ. Eng. Asst., Dept. of Water & Power, Box 3669 Terminal Annex, Los Angeles 54, Calif. (Apr. '56) *D*

Fenger, Albert C., Asst. City Engr., Greenville, Miss. (Apr. '56) *M*

(*Continued on page 92 P&R*)

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Fisher, George R., City Engr. & Water Supt., Pasco, Wash. (Apr. '56) *M*

Fisk, Charles C., Administrative Asst., Water Board, City & County Bldg., Denver, Colo. (Apr. '56) *MRD*

Flucard, Oren A., Supt., Water & Sewer Dept., Box 368, Gravette, Ark. (Apr. '56) *MP*

Fowler, R., Administrative Asst., Water Dept., 323 County-City Bldg., Seattle, Wash. (Apr. '56)

Freeman, Arthur C., III, Water & Sewer Engr., Dept. of Public Works, Norfolk, Va. (Apr. '56)

Frison, Paul, Director, Ozone Dept., Societe Trailigaz, 55, Ave. A. France, Colombes (Seine), France (Apr. '56)

Fuehrer, Patricia L. (Miss), Jr. San. Engr., State Dept. of Health, Bureau of San. Eng., 413 First National Bank Bldg., Greensburg, Pa. (Jr. M. Apr. '56) *RP*

Gamble, William B., Jr., Vice-Pres., American Water Works Service Co., Inc., 3 Penn Center Plaza, Philadelphia 2, Pa. (Apr. '56) *M*

Gammeter, Edward T., Maint. Supt., Water Dept., 4600 McRee, St. Louis, Mo. (Apr. '56) *MD*

Gillespie, M. W., Dist. Mgr., Pittsburgh Pipe Cleaner Co., 434 Primos Ave., Folcroft, Pa. (Apr. '56) *D*

Glenn, Charles A., Gen. Sales Mgr., Lynchburg Foundry Co., Lynchburg, Va. (Apr. '56) *MRPD*

Green, Severn T., Sales Engr., De Laval Steam Turbine Co., 206 Daniel Bldg., Tulsa, Okla. (Apr. '56) *PD*

Greensboro Public Service Dept., W. M. Lybrook, Director, Rm. 301, Municipal Bldg., Greensboro, N.C. (Munic. Sv. Sub. Apr. '56) *MRPD*

Grimes, Paschal, City Engr., Memorial Bldg., Chanute, Kan. (Apr. '56) *M*

Gulf Seal Corp., Jarl R. Sahol, Vice-Pres., 1522 Melrose Bldg., Houston 2, Tex. (Assoc. M. Apr. '56)

Hanson, Harry G., Director, US Public Health Service, Robert A. Taft San. Eng. Center, 4676 Columbia Pkwy., Cincinnati 26, Ohio (Apr. '56)

Hart, Roger L., Water Works Operator, Ravenna Arsenal Inc., Apco, Ohio (Apr. '56) *PD*

Hayward, Homer J., Assoc. Engr., Ayres, Lewis, Norris & May, 500 Wolverine Bldg., Ann Arbor, Mich. (Apr. '56) *RPD*

Heydman, Irvan W., Maint. Foreman, Hefner Filtration Plant, Oklahoma City, Okla. (Apr. '56) *MD*

Hildenbrand, Leo R., Mgr., Water Dept., Bismarck, N.D. (Apr. '56) *M*

Hollifield, Howard, Jr., Div. Commercial Mgr., Burns & Roe of Michigan, Inc., Book Bldg., Detroit 26, Mich. (Apr. '56) *MRPD*

Hopp, Carl W., Sales Mgr., American Pipe & Constr. Co., 518 N.E. Columbia Blvd., Portland 11, Ore. (Apr. '56) *D*

Hunt, James F., Sales Repr., American Cyanamid Co., 3333 Wilkinson Blvd., Charlotte, N.C. (Apr. '56) *RPD*

Hurlbut, Robert A., Gen. Supt., City of Miller, Box 374, Miller, S.D. (Apr. '56) *MRD*

Johnson, A. S., Mgr., Water & Sewage Works, Earle, Ark. (Apr. '56) *M*

Johnson, Albert H., Public Health & Safety Div., Tennessee Valley Authority, Chattanooga, Tenn. (Apr. '56) *RP*

Johnson, Robert W., Chemist, St. Louis County Water Co., 8390 Delmar Blvd., St. Louis 24, Mo. (Apr. '56) *P*

Johnson, Stanford E., Foreman, Water Dept., Sturgeon Bay Utilities, Box 239, Sturgeon Bay, Wis. (Apr. '56) *MD*

Keady, G. H., Supt., Public Works, Pittsfield, Ill. (Apr. '56) *M*

Kiomento, Domenic; *see* Brighton (Mich.)

Kleine, Charles F., Assoc. Hydr. Engr., State Dept. of Public Works, Div. of Water Resources, 1170 North St., Sacramento, Calif. (Apr. '56) *RP*

Kraft, Gerald G., Gen. Mgr., Kraft Chem. Co., 917 W. 18th St., Chicago 8, Ill. (Apr. '56)

Lewin, Alan B.; *see* Lewin-Mathes Co.

Lewin-Mathes Co., Alan B. Lewin, Vice-Pres., 111 Chouteau Ave., St. Louis 2, Mo. (Assoc. M. Apr. '56)

Loudoun Water Corp., George H. Engeman, Secy., Sterling, Va. (Corp. M. Apr. '56) *M*

Lucas, Perry, Water Supt., Lakewood Water & Power Co., 2965 E. Carson St., Lakewood, Calif. (Apr. '56) *PD*

Luis, Mantilla F., Dean, Faculty of San. Eng., Univ. of Engineering, Lima, Peru (Apr. '56) *PD*

Lum, Richard W. K., Civ. Engr., Board of Water Supply, Box 3410, Honolulu 1, Hawaii (Apr. '56) *RD*

Lund, William J., Water Works Operator, 211—1st St., E., Canby, Minn. (Apr. '56) *MD*

Lybrook, W. M.; *see* Greensboro (N.C.) Public Service Dept.

Lyon, Charles R., Design Engr., R. Ruskin Fisher & Assocs., 17962 Aurora Ave., Richmond Highlands, Wash. (Apr. '56) *PD*

Mack, J. L., Jr., Vice-Pres., White Star Sales Corp., Box 958, Jacksonville, Fla. (Apr. '56) *D*

Mann, R. D., Sales Engr., Fischer & Porter Co., 22750 Woodward, Detroit 20, Mich. (Apr. '56) *P*

Maper, John L., Regional Sales Mgr., Sparling Meter Co., 225 N. Temple City Blvd., El Monte, Calif. (Apr. '56) *R*

Markoff, Daniel, Corrosion Engr., D. Markoff Assocs., Box 541, Hermosa Beach, Calif. (Apr. '56) *MD*

McBride, H. D.; *see* Monolith Portland Midwest Co.

McDonald, Thomas L., Sr. San. Chemist, Safety Control Sec., 3300 E. Cheltenham Pl., Chicago, Ill. (Apr. '56) *P*

McKim, Robert D., Civ. Engr., Black & Veatch, 4706 Broadway, Kansas City, Mo. (Apr. '56) *MRP*

McMullen, Robert E.; *see* Clayton County (Ga.) Water Authority

Meginnis, Charles E., Dist. Sales Mgr., Davison Chem. Co., Div. of W. R. Grace & Co., 19 S. LaSalle St., Chicago 3, Ill. (Apr. '56)

Mejias, Diego B., Secy., Inst. Nacional de Obras Sanitarias, Las Mercedes Bldg., Caracas, Venezuela (Apr. '56)

Michael, W. F., Sales Engr., Dorr-Oliver-Long Ltd., 26 St. Clair Ave. E., Toronto, Ont. (Apr. '56)

Milinowski, Arthur S., Assoc. Engr., Buck, Seifert & Jost, 112 E. 19th St., New York 3, N.Y. (Apr. '56) *R*

Miller, Perry E., Chief, Industrial Waste Sec., State Board of Health, 1330 W. Michigan, Indianapolis, Ind. (Apr. '56) *RP*

Moeller, Delmer H., Graduate Student, Univ. of Wisconsin, Hydr. & San. Lab., Madison, Wis. (Jr. M. Apr. '56) *P*

Monolith Portland Midwest Co., H. D. McBride, Mgr., Oxide Div., 208 Tramway Bldg., Denver 2, Colo. (Assoc. M. Apr. '56)

Moore, William B., Civ. Engr., Carey & Kramer, 1917—1st Ave., Seattle 1, Wash. (Apr. '56) *P*

Moulton, Howard A., Field Supt., Water Distr., Dept. of Water & Power, Box 3669 Terminal Annex, Los Angeles 54, Calif. (Apr. '56) *MRPD*

Murphy, Raymond F., Partner, Murphy & Hastings, 650 El Camino Real, Redwood City, Calif. (Apr. '56) *PD*

Nair, DamodaranAsan Appukuttan, Public Health Engr., Travancore-Cochin State, Trivandrum 1, India (Apr. '56) *PD*

Naylor, Earl H., Supt., Water Dept., Board of Water Comrs., Burlington, N.J. (Apr. '56) *MRPD*

Nicol, James S., Dist. Sales Mgr., The Permutit Co., 1111 Nicolet Ave., Minneapolis, Minn. (Apr. '56) *P*

Owers, Leonard E., San. Engr., Provincial Dept. of Health, 807 Richmond St. W., Toronto, Ont. (Apr. '56)

Palmer, James A., Fuels Engr., Libbey-Owens-Ford Glass Co., 1701 E. Broadway, Toledo 5, Ohio (Apr. '56) *RP*

Parry, Irvin H., Field Supervisor, Watertower Paint & Repair Co., Inc., Box 306, Clear Lake, Iowa (Apr. '56) *MP*

Pebworth, W. B., Hersey Mfg. Co., 2108 Brewton St., Montgomery, Ala. (Apr. '56)

Peletz, Harold; *see* Peletz Co.

Peletz Co., Inc., Harold Peletz, Pres., 475 W. College Ave., Santa Rosa, Calif. (Assoc. M. Apr. '56)

Perlick, Edward E., Sales Engr., Allis-Chalmers Mfg. Co., Rm. 4110, 135 S. LaSalle St., Chicago 3, Ill. (Apr. '56) *PD*

Press, Emil A., Deputy Supt., Office of Planning, Design & Eng., District of Columbia Dept. of San. Eng., District Bldg., Washington, D.C. (Apr. '56) *MD*

(*Continued on page 94 P&R*)

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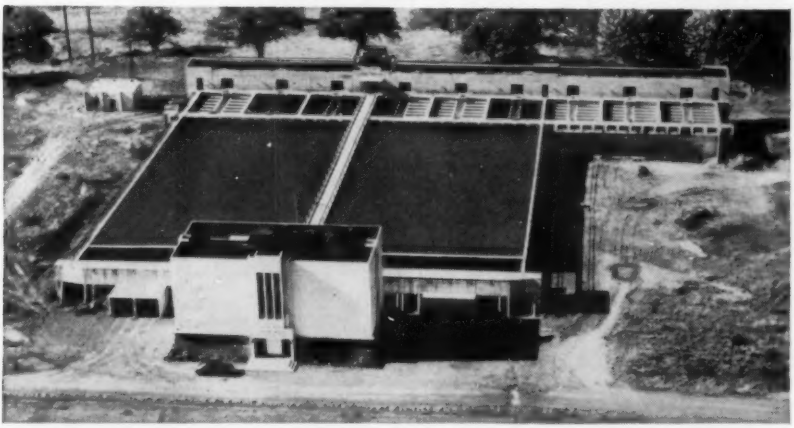

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(*Continued from page 92 P&R*)

Proudfit, Donald P., Project Engr., Black & Veatch, 4706 Broadway, Kansas City, Mo. (Apr. '56)

Reeder, Harvey W., Civ. Eng. Asst., Dept. of Water & Power, Box 3669 Terminal Annex, Los Angeles 54, Calif. (Apr. '56) *RPD*

Rieser, Douglas A., Chief Engr., T. W. Snow Constr. Co., 10 W. Wilson St., Batavia, Ill. (Apr. '56)

Rogers, Ernest H., San. Engr., Water Dept., 323 County-City Bldg., Seattle, Wash. (Apr. '56) *RP*

Ryan, Andrew L., Supt., Water Dept., Fair Lawn, N.J. (Apr. '56) *PD*

Sahol, Jarl R.; *see* Gulf Seal Corp.

Schoell, William D., Partner, Schoell and Madson, 8134 Minnetonka Blvd., St. Louis Park 26, Minn. (Apr. '56) *MRPD*

Scott, G. L. C., Asst. to Sales Mgr., American Pipe & Constr. Co., 518 N.E. Columbia Blvd., Portland 11, Ore. (Apr. '56) *D*

Shea, George H., Chief Engr., Water Works, 20 Commercial St., Holyoke, Mass. (Apr. '56) *MRPD*

Silva, Edward, Field Supt., Dept. of Water & Power, Box 3669 Terminal Annex, Los Angeles 54, Calif. (Apr. '56) *D*

Simmons, Emmett G.; *see* Engine & Equipment Co.

Skodje, Marvin T., Assoc. Public Health Engr., Div. of Environmental Sanitation, State Dept. of Health, Univ. Campus, Minneapolis, Minn. (Apr. '56) *P*

Spear, Clyde C., Supt., Water Works, Delphi, Ind. (Apr. '56) *MD*

Spitz, Frederick M.; *see* Chemical Testing Corp.

Sprague, C. B., Plant Engr., Columbian Carbon Co., Box 1459, Conroe, Tex. (Apr. '56) *RP*

Staben, Fred, Asst. Supt., Water Works, Lake Front, Waukegan, Ill. (Apr. '56) *MPD*

Stoffel, L. L., Sales Repr., Johns-Manville Sales Corp., Pipe Div., 1642 Sunfield St., Madison 4, Wis. (Apr. '56) *D*

Symons, Kelvin, San. Engr., Provincial Dept. of Health, 807 Richmond St. W., Toronto, Ont. (Apr. '56)

Thompson, Julius F., Jr., Supt., Filter Plant, Box 365, Graham, N.C. (Apr. '56) *MP*

Thompson, William L., City Administrator, 101 Holloway St., Hurst, Tex. (Apr. '56) *MRP*

Timme, Paul A., Salesman, Badger Meter Mfg. Co., 211 Beckley St., Wernersville, Pa. (Apr. '56) *D*

Tolleson, William W., Vice-Pres., Lee Constr. Co., Box 3608, Charlotte, N.C. (Apr. '56) *P*

Toppari, P. M., Chief Chemist, Rayonier, Inc., Grays Harbor Div., Hoquiam, Wash. (Apr. '56) *RP*

Turetsky, Isadore, Sr. Elec. Pumping Plant Operator, Dept. of Water & Power, 410 Ducommun St., Los Angeles 12, Calif. (Apr. '56) *R*

Tyler Pipe & Foundry Co., Earl C. Bradley Jr., Mgr., Utilities Dept., Box 2027, Tyler, Tex. (Assoc. M. Apr. '56)

Umbaugh, H. J., C.P.A., 218 W. LaPorte St., Plymouth, Ind. (Apr. '56) *M*

Van Buskirk, D. E., Production Engr., Water Div., Houston, Tex. (Apr. '56)

Van Sickle, Norman E., Cons. Engr., Box 733, Hoyt Lakes, Minn. (Apr. '56) *RPD*

Volpe, Thomas J., Sr. Civ. Eng. Asst., Los Angeles County Flood Control Dist., 2250 Alcazar St., Los Angeles 33, Calif. (Apr. '56) *RP*

Walden, Herbert C., Supt., Water Plant, Broadview Park Water Co., 4400 Peters Rd., Fort Lauderdale, Fla. (Apr. '56) *MP*

Watlington, James E., Jr., Cons. Engr., Austin Brockenbrough & Assocs., 202 E. Cary St., Richmond 19, Va. (Apr. '56) *RPD*

Watson, John M., Asst. Dist. Mgr., Permutit Co. of Canada Ltd., 5500 Royalmount Ave., Montreal, Que. (Apr. '56) *P*

Weeber, W. Keith, San. Engr., State Dept. of Public Health, 121—4th Ave., Rock Island, Ill. (Apr. '56) *P*

Welch, Ronald J., Civ. Eng. Asst., Dept. of Water & Power, 316 W. 2nd St., Los Angeles, Calif. (Apr. '56) *RD*

Welter, Woodrow Leo, Asst. Supt., Water Dept., 30 Gerald Ave., Highland Park 3, Mich. (Apr. '56) *M*

Wentworth, Earl A., Sr. Foreman, Meter Shop, Water Distr., 1970 B St., San Diego, Calif. (Apr. '56) *MD*

West, Robert H., 329 S. 51st Ave., Bellwood, Ill. (Apr. '56)

Wilfong, Stanley A., Engr., Dept. of Water & Power, 316 W. 2nd St., Los Angeles, Calif. (Apr. '56)

Wilkinson, George, Town Clerk, Aurora, Ont. (Apr. '56)

Williams, Harry D., Jr., Student, Univ. of Wisconsin, 1820 Baird St., Madison, Wis. (Jr. M. Apr. '56) *MP*

Wise, Robert S., Mgr., Tech. Service, National Aluminate Corp., 6216 W. 66th Pl., Chicago 38, Ill. (Apr. '56) *P*

Wolfe, William D., Mgr., Welded Pipe Sales, Armco Drainage & Metal Products, 2600—7th St., Berkeley, Calif. (Apr. '56) *D*

Zimmerman, Jeremy C., Design Engr., Consoer, Townsend & Assocs., 360 E. Grand Ave., Chicago 11, Ill. (Apr. '56) *P*

Zollt, Bernard E., Chief Chemist, Metropolitan Refining Co., Inc., 50–23—23rd St., Long Island City 1, N.Y. (Apr. '56) *MRPD*

Zwinge, Gilbert D., Water Supt. & Mgr., 145 Robertson Blvd., Chowchilla, Calif. (Apr. '56) *D*

EMGAMY Honor Roll

Listed below are the AWWA members who have contributed to "Every-Member-Get-A-Member Year" by getting members for the Sections shown, in the period Apr. 1–May 31. (Previous EMGAMblers were listed in the April 1956 issue of Willing Water.) *Numbers in parentheses indicate more than one member enrolled.*

Alabama-Mississippi

Eshelman, J. W.
Gilmore, W. H.
Perry, C. T.
Teunisson, J. H., Jr.

California

Alden, Charles
Banks, H. O.
Bascom, C. V.
Cassidy, H. J. (4)
Dye, R. D.
Foster, E. R.
Hersom, Ernie
Hollepeter, L. T.
Hurlbut, W. W.
Jerauld, H. F.
Keirn, K. A.
Kells, King
Kern, Kenneth
Lane, P. H.
McCabe, Joseph
Mylander, H. A.
Ongerth, H. J.
Owen, M. H.
Parbury, C. B.
Roberts, H. G.
Seeger, W. R.
Volk, K. Q.
Wallace, K. K. (2)
Webber, W. L.
Weight, W. O.
Whitman, E. R.
Youngquist, A. C.

Canadian

Berry, A. E. (6)
Lafreniere, T. J.
Meindl, J. A.

Chesapeake

Haywood, R. W.
Joseph, A. B.

Florida

Earls, C. E. (2)

Illinois

Cella, J. R.
Domke, L. C.
Foley, M. H.
Gerstein, H. H.
Gordon, F. G.
Hallden, O. S. (2)
Holmes, J. A.
Howson, L. R.
Johnson, P. H.
Kuehn, N. H., Jr.
Larson, T. E. (2)
Mrva, A. E.
Pigg, W. L.
Powers, C. B.
Scarritt, E. W.
Soderberg, A. L.
Van Meter, L. R.
Whisler, J. H.

(*Continued on page 96 P&R*)

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AWWA WATER RATES MANUAL

The report of the AWWA Committee on Water Rates published in the March 1954 JOURNAL is now available in reprint form as part of the AWWA "Water Rates Manual." Bound in with the report are generous excerpts from such classic publications on the subject of water rates as the Metcalf, Kuichling, and Hawley paper of 1911 on fire protection charges and the reports of NEWWA and AWWA committees chairmanned by Allen Hazen, as well as a table of cost indexes, an outline for allocation of plant and expenses, and other valuable supplementary material.

64 pages **$1.00**

American Water Works Association

2 Park Ave., New York 16, N. Y.

(Continued from page 94 P&R)

Indiana
Briggs, H. C.
Finch, L. S.
Gordon, Jack (3)
Rademacher, J. M.

Iowa
Ellis, R. O. (2)
Hail, J. J.

Kansas
Badley, H. W.
Chandler, R. L.
Collar, L. H.
Dominick, W. H.
Ellis, R. O.
Janzen, H. A.
Mounsey, R. J. (3)
Pelton, G. D. (2)
Scott, G. R.
Trammell, C. R.

Kentucky-Tennessee
Brickey, M. L.
French, C. H.
Henshaw, J. A.
Kochtitzky, O. W.
Sanders, R. N.

Michigan
Ayres, L. E. (2)
Borchardt, J. A.
Coleman, E. L.
Hinebrook, V. L.
Kimble, W. V.
Vander Velde, T. L. (4)
Vogt, J. E.

Missouri
Beatie, R. H.
Buettner, C. F.
Gibbs, W. R.
Hartung, H. O. (2)
Lischer, V. C.
Ramsey, J. B.
Serkes, Meyer
Skinker, T. J.
Trammell, C. R.

Montana
Clarkson, A. W.
Cronin, R. G.

Nebraska
Detweiler, J. C. (6)

New England
Karalekas, P. C.
Root, D. A.

New Jersey
Hoffer, H. A.
Jost, C. F.
Knapp, Frank

New York
Becker, Morris
Blanchard, Kimball
Clark, R. S. (2)
Diven, J. M. (3)
Elder, F. X.
Frank, J. A.
Pope, W. J.

North Carolina
Ameen, J. S.
Phillips, R. S. (2)
Rawlins, G. S.
Thompson, J. F., Sr.
Young, E. G. (2)

North Central
Bach, C. M.
Carl, C. E.
Haendel, W. P.
Huset, E. A., Jr. (2)
Kalda, D. C. (2)
LeVan, H. A., Jr.
Petersen, R. W.
Yegen, William

Ohio
Cook, L. F.
Frederick, V. W.
Hamlin, G. W.
Howell, Robert
LaDue, W. R. (2)
Myers, Howard
Poston, H. W.
Reed, A. M.
Reed, R. E.

Pacific Northwest
Bogan, R. H.
Braun, P. J. (5)
Brett, L. E. (2)
Dunstan, G. H.
Marqueling, H. E.
Peterson, R. B.
Ramey, J. C.
Sylvester, R. O.
Taylor, H. D.
Westgarth, W. C.

Pennsylvania
Arnold, G. E. (2)
Carlock, H. J.
Daly, W. J.
Glace, L. M., Jr.
Hicks, H. K.
Krum, H. J.
Morgan, L. S. (2)
Murdoch, J. H., Jr. (2)
Robinson, Harold

Rocky Mountain
Burgess, John
Lynch, J. G.

Southeastern
deJarnette, N. M.

Southwest
Amsbary, F. C., Jr.
Bankston, L. V. (5)
Barngrover, Lester (4)
Black, N. W.
Connell, C. H.
Creel, H. G.
Davis, Wayne
Deal, J. C.
Ferris, T. C. (2)
French, C. H. (9)
Gentry, Ted (3)
Hill, Underwood
Kellogg, G. T.
Moehlmann, Roger
Moutrey, C. E.
Reynolds, J. S.
Rice, D. R.
Sanders, R. N. (6)
Sellers, J. L.
Street, H. R.
Taylor, F. S. (2)
Wright, A. L.

Virginia
Brockenbrough, Austin, Jr.
Ferguson, J. B.
Fitzgerald, R. W.
Kavanagh, J. P.
Nordby, R. C.
Odor, W. R.
Pharr, J. M.

Wisconsin
Leistikow, W. F.
Rohlich, G. A. (2)
Smith, L. A.
Vander Velde, T. L. (2)

Foreign
Gravel, C. E.
Hurtado, J. R. (2)
Young, E. F.

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- QUICK AND FEW REPAIRS

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"THE METER YOU CAN COUNT ON"
WATER METER

MANUFACTURED BY
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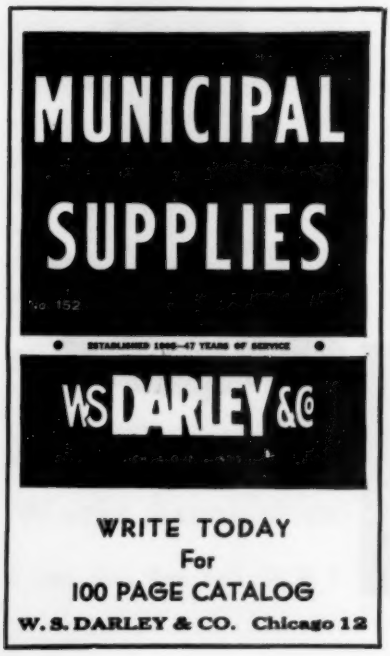
FORT WORTH, TEXAS

(Continued from page 50 P&R)

dence in a handy fishbowl (the piscatorial, not the pilsener, type). The evidence was a snake which appeared in Power's tub while he was taking a bath and bit him in the big toe. Fished out of the water, what had seemed 10 ft long dwindled to 18 in., but a snake it was, and apparently it had entered the tub through the drain pipe when Power pulled the plug. Only a week later in St. Louis, during AWWA's Diamond Jubilee, still more snakes were said to be seen, but nary a one was captured.

What next to whet our thirst—for what?

A. William Fraser and Clarence S. Wentworth have been appointed commercial vice-presidents of Worthington Corp., Harrison, N.J. Both men are regional sales managers, Mr. Fraser with headquarters in Chicago and Mr. Wentworth in Cleveland.



Penn Instrument Div. of Burgess-Manning Co., Philadelphia, has appointed Flow Control Corp., Tampa, as its Florida representative.

Morton Salt Co. has opened a multi-million-dollar laboratory designed especially for the conduct of research projects related to salt and its usage. Located at Woodstock, Ill., near Chicago, the laboratory is headed by Dr. Robert L. Frank.

Western offices have been established at Hot Springs, S.D., by Carl A. Bays & Assocs., Urbana, Ill., industrial consultants. Manager of the branch is Gordon W. Prescott, who has been representing the firm at Hot Springs.

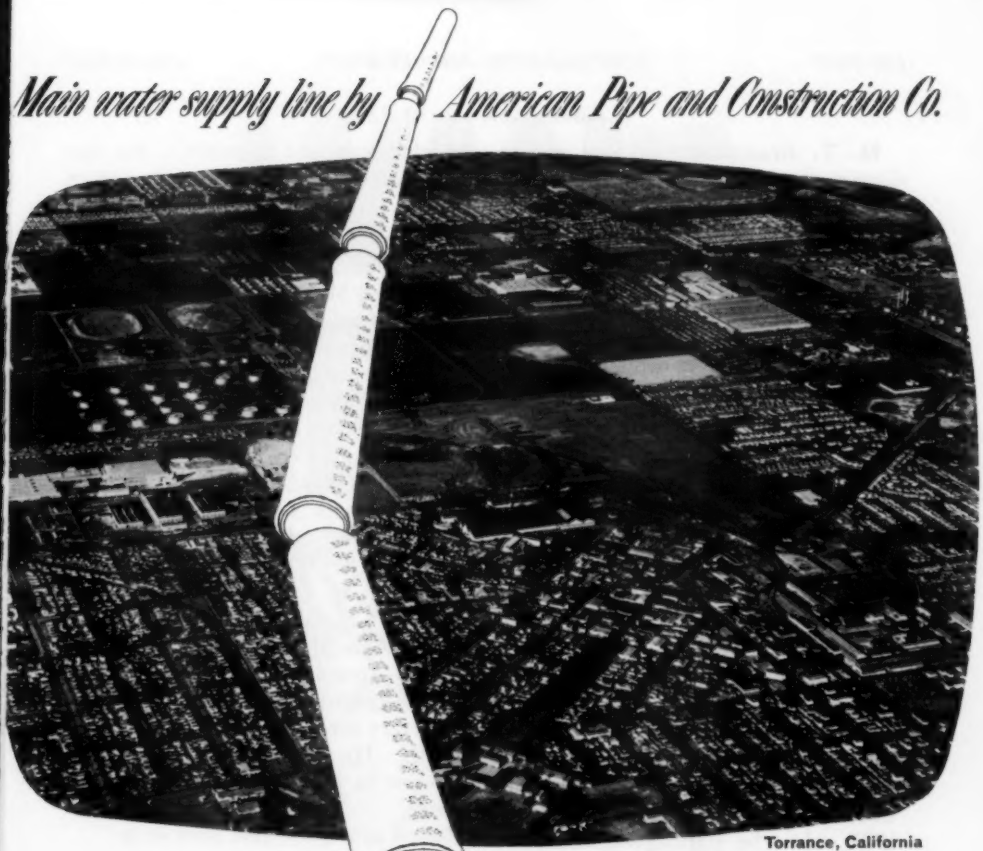
Bartow's Golden Age is what they're calling it out at Iowa City, Iowa, where Dr. Edward Bartow, professor-emeritus at the University of Iowa, has been golden anniversarying all over the place. Latest bullion to arrive was the "Golden Doctor's Diploma" of the University of Gottingen in Germany, traditionally conferred by the university 50 years after the recipient receives his doctorate. Actually, Dr. Bartow's Ph.D. degree was dated Oct. 25, 1895, but life at Gottingen in 1945 was such as to necessitate postponing the golden diplomacy until now.

In June still another golden anniversary rolled around, when Dr. Bartow completed 50 years as an AWWA member. He really celebrated that date a little in advance by attending and helping AWWA salute its Diamond Jubilee in St. Louis in May—just 35 years after he took office as the Association's president.

"It pays to live a long time," says our good doctor . . . in gold, no less!

(Continued on page 100 P&R)

Main water supply line by American Pipe and Construction Co.



Torrance, California

'Backbone of the System'

says **Angus E. McVicar**
Torrance, California
Water Superintendent

Concentration of industry and a population that has tripled since 1950 posed a serious water problem for the city of Torrance, California. Although ample water was available from the Metropolitan Water District of Southern California, only one of the city's three water systems was tied into the District's supply line and the city's principal growth was in two areas supplied independently by wells and reservoirs.

Solution was the installation of 22,000′ of 24" American Concrete Cylinder Pipe which coordinated all three systems and allowed distribution of Metropolitan water wherever desired. "The new line is the backbone of the system," says Angus E. McVicar, Torrance water superintendent. The installation of American Concrete Cylinder Pipe now provides Torrance with a permanent, maintenance-free water supply line of sustained carrying capacity and virtually limitless life.

American Pipe makes available 49 years of experience and extensive production facilities to help solve any water supply problem. There is a type of American Pipe to meet any requirement. Write or phone for complete information.

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American PIPE AND CONSTRUCTION CO.

CONCRETE PIPE FOR MAIN WATER SUPPLY LINES, STORM AND SANITARY SEWERS, SUBAQUEOUS LINES

(*Continued from page 98 P&R*)

M. T. Bramman has been named district manager for valves by S. Morgan Smith Co. of York, Pa. He will cover the Pacific Coast area.

New Jersey applicants for supply or treatment plant operators' licensing examinations must file by Nov. 10, 1956. Forms and information can be obtained from Bureau of Examination & Licensing, State Dept. of Health, State House, Trenton 25, N.J.

Cochrane Corp., Philadelphia, has announced that its subsidiary, Cochrane Water Conditioning Ltd., is now active in Canada, with general offices in Toronto under Murray Dobier, vice-president and general manager.

Norman J. Howard, retired director of water purification and sewage treatment, Toronto, Ont., died early in May 1956. Born in Ireland, he received an arts degree from Oxford University and studied medicine until 1907, when he joined the laboratory staff of William Crookes and James Dewar and worked on the water supplies of London, India, and Egypt. In 1909 he went to work for the newly formed Metropolitan Water Board of London. Two years later he accepted the position of assistant chemist and bacteriologist at a Toronto filter plant then nearing completion. The following year he was given full charge of the city's water supply laboratories. In 1927 he was named director of purification, a position he held until his retirement in 1954. During his tenure, Mr. Howard played an important part in the advancement of water treatment.

Joining AWWA in 1920, he was president in 1941 and was named an Honorary Member in 1952. He received the Fuller Award in 1943 from the Canadian Section, which he served as chairman and director. He was active on many Association committees.

Leon A. Smith, superintendent, water works and sewerage, Madison, Wis., died May 10, 1956, at St. Louis, while attending the AWWA Conference. He was 66. Born at Elgin, Ill., in 1890, he was a graduate of the University of Wisconsin. In 1912 he became assistant city engineer at Madison, and in 1916 he was appointed water superintendent. By planning ahead for its water needs over the years, he helped Madison grow from a town of 25,000 to a city of more than 100,000.

A Life Member of AWWA (joined in 1916), Mr. Smith received the Fuller Award in 1939 from the Wisconsin Section, which he served as secretary for nearly 30 years, until his death. He was also chairman of the Section in 1925 and served as director twice.

A. Baldwin Wood, general superintendent of the New Orleans Sewerage & Water Board, died May 10, 1956, of a heart attack while sailing alone on his boat near his Biloxi, Miss., home. He was 77 years old. A native of New Orleans, he was graduated from Tulane University in 1899. In that year he went to work as assistant manager of drainage for the New Orleans Drainage Commission, which later merged with the water board to form the present Sewerage & Water Board. He maintained his connection with the board throughout his life (since 1938 as general superintendent), although he had an active private consulting practice and was also engaged in the design and marketing of pumps. He had been an AWWA member since 1939.

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Aluminum Alloy Tapping Machine

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- A third less weight to handle
- Requires less time
- Requires less working room
- Easier to operate
- No disassembly to insert stop
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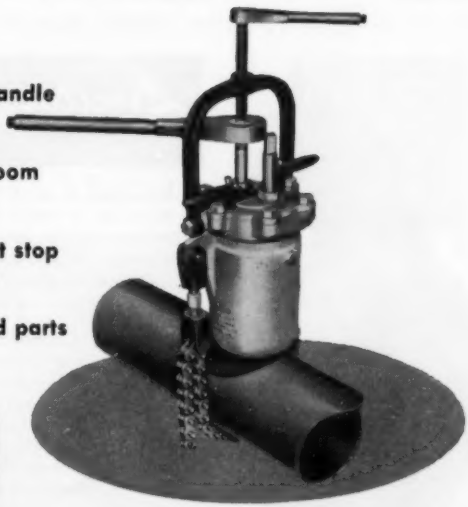
- Complete with chest



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Join the A. W. W. A. HAYS is one of the eleven Charter Members of the Manufacturers Section of the American Water Works Association.

HAYS

WATER WORKS PRODUCTS

HAYS MANUFACTURING CO.

ERIE, PA.

Index of Advertisers' Products

Activated Carbon:
Industrial Chemical Sales Div.
Permutit Co.

Activated Silica Generators:
Omega Machine Co. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

Aerators (Air Diffusers):
American Well Works
Carborundum Co.
General Filter Co.
Infilco Inc.
Permutit Co.
Walker Process Equipment, Inc.

Air Compressors:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
Morse Bros. Mchy. Co.

Alum (Sulfate of Alumina):
American Cyanamid Co., Heavy Chemicals Dept.
General Chemical Div.

Ammonia, Anhydrous:
General Chemical Div.
John Wiley Jones Co.

Ammoniators:
Fischer & Porter Co.
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Co., Inc.

Brass Goods:
American Brass Co.
M. Greenberg's Sons
Hays Mfg. Co.
Mueller Co.

Calcium Hypochlorite:
John Wiley Jones Co.

Carbon Dioxide Generators:
Infilco Inc.
Walker Process Equipment, Inc.

Cathodic Protection:
Electro Rust-Proofing Corp.
Harco Corp.

Cement, Portland:
Monolith Portland Midwest Co.

Cement Mortar Lining:
Centriline Corp.

Chemical Feed Apparatus:
Cochrane Corp.
Fischer & Porter Co.
Graver Water Conditioning Co.
Infilco Inc.
F. B. Leopold Co.
Omega Machine Co. (Div., B-I-F Industries)
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries)
Ross Valve Mfg. Co.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Chemists and Engineers:
(See Professional Services)

Chlorination Equipment:
Builders-Providence, Inc. (Div., B-I-F Industries)
Everson Mfg. Corp.
Fischer & Porter Co.
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

Chlorine Comparators:
Klett Mfg. Co.
Wallace & Tiernan Inc.

Chlorine, Liquid:
John Wiley Jones Co.
Wallace & Tiernan Inc.

Clamps and Sleeves, Pipe:
James B. Clow & Sons
Dresser Mfg. Div.
M. Greenberg's Sons
Mueller Co.
Rensselaer Valve Co.
Skinner, M. B., Co.
A. P. Smith Mfg. Co.
Smith-Blair, Inc.
Trinity Valley Iron & Steel Co.

Clamps, Bell Joint:
James B. Clow & Sons
Dresser Mfg. Div.
Skinner, M. B., Co.

Clamps, Pipe Repair:
James B. Clow & Sons
Dresser Mfg. Div.
Skinner, M. B., Co.
Trinity Valley Iron & Steel Co.

Clarifiers:
American Well Works
Chain Belt Co.
Cochrane Corp.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Infilco Inc.
Permutit Co.
Walker Process Equipment, Inc.

Type SM with Worm Gear Drive on Butterfly Valve.

Limitorque® VALVE CONTROLS

From coast to coast, hundreds of LimiTorque Controls are in service in central stations and power plants for automatic or push-button operation of valves up to 120 inch diameter. Why is acceptance so widespread? Because LimiTorque Operators are designed to provide dependable, safe and sure valve actuation at all times.

LimiTorque is self-contained and is applicable to all makes of valves. Any available power source may be used to actuate the operator: Electricity, water, air, oil, gas, and are readily adapted to Microwave Control.

A feature of LimiTorque is the torque limit switch which controls the closing thrust on the valve stem and prevents damage to valve operating parts.

Write for Catalog L-550

Philadelphia Gear Works, Inc.
ERIE AVE. and G STREET, PHILADELPHIA 34, PA.
NEW YORK • PITTSBURGH • CHICAGO • HOUSTON • LYNCHBURG, VA. • BALTIMORE • CLEVELAND

Armco Steel Water Pipe

goes in fast

Long lengths of Armco Steel Pipe save you time and money when laying a water line. Supplied in lengths up to 50 feet, Armco Pipe goes in fast, requires fewer joints. Labor costs are low.

By specifying Armco Steel Pipe you can select from a wide range of sizes— 6-to 36-inch diameter, 9/64-to 1/2-inch wall thickness—to meet your water line requirements exactly. Tees, wyes, elbows and other fittings can be made to order.

Armco Water Pipe is coated to AWWA specifications. A smooth spun enamel lining prevents tuberculation and assures continued high flow capacity. For more data on Armco Welded Steel Pipe write to: Armco Drainage & Metal Products, Inc., Welded Pipe Sales Division, 3086 Curtis Street, Middletown, Ohio. Subsidiary of Armco Steel Corporation. In Canada write: Guelph, Ontario.

WHEREVER WATER FLOWS, STEEL PIPES IT BEST

ARMCO

50-foot lengths of Armco Steel Water Pipe are easily handled using standard equipment.

Armco Welded Steel Pipe

Meets AWWA Specifications

Condensers:
Alco Products, Inc.
Allis-Chalmers Mfg. Co.
United States Pipe & Foundry Co.

Contractors, Water Supply:
Layne & Bowler, Inc.

Controllers, Liquid Level, Rate of Flow:
Builders-Providence, Inc. (Div., B-I-F Industries)
Fischer & Porter Co.
Foxboro Co.
General Filter Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Simplex Valve & Meter Co.

Copper Sheets:
American Brass Co.
Copper Sulfate:
General Chemical Div.
Phelps Dodge Refining Corp.
Tennessee Corp.

Corrosion Control:
Alco Products, Inc.
Calgon, Inc.
Philadelphia Quartz Co.

Couplings, Flexible:
DeLaval Steam Turbine Co.
Dresser Mfg. Div.
Philadelphia Gear Works, Inc.

Diaphragms, Pump:
Dorr-Oliver Inc.
Morse Bros. Mchy. Co.
Southern Pipe & Casing Co.

Engines, Hydraulic:
Ross Valve Mfg. Co.

Engineers and Chemists:
(See Professional Services)

Feedwater Treatment:
Allis-Chalmers Mfg. Co.
Calgon, Inc.
Cochrane Corp.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries)

Ferric Sulfate:
Tennessee Corp.

Filter Materials:
Anthracite Equipment Corp.
Carborundum Co.
General Filter Co.
Infilco Inc.
Johns-Manville Corp.
Northern Gravel Co.
Permutit Co.
Carl Schleicher & Schuell Co.
Stuart Corp.

Filters, incl. Feedwater:
Cochrane Corp.
Dorr-Oliver Inc.
Graver Water Conditioning Co.
Infilco Inc.
Morse Bros. Mchy. Co.
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries)
Roberts Filter Mfg. Co.
Ross Valve Mfg. Co.

Filters, Membrane (MF):
Millipore Filter Corp.
Carl Schleicher & Schuell Co.

Filtration Plant Equipment:
Builders-Providence, Inc. (Div., B-I-F Industries)
Chain Belt Co.
Cochrane Corp.
Filtration Equipment Corp.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
F. B. Leopold Co.
Omega Machine Co. (Div., B-I-F Industries)
Permutit Co.
Roberts Filter Mfg. Co.
Simplex Valve & Meter Co.
Stuart Corp.
Wallace & Tiernan Inc.

Fittings, Copper Pipe:
Dresser Mfg. Div.
M. Greenberg's Sons
Hays Mfg. Co.
Mueller Co.

Fittings, Tees, Ells, etc.:
Alco Products, Inc.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Crane Co.
Dresser Mfg. Div.
M & H Valve & Fittings Co.
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co
R. D. Wood Co.

Flocculating Equipment:
Chain Belt Co.
Cochrane Corp.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Infilco Inc.
F. B. Leopold Co.
Permutit Co.
Stuart Corp.

Fluoride Chemicals:
American Agricultural Chemical Co.

Fluoride Feeders:
Fischer & Porter Co.
Omega Machine Co. (Div., B-I-F Industries)
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Co., Inc.

Furnaces:
Jos. G. Pollard Co., Inc.

Gages, Liquid Level:
Builders-Providence, Inc. (Div., B-I-F Industries)
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Gages, Loss of Head, Pressure & Vacuum, Rate of Flow, Sand Expansion:
Builders-Providence, Inc. (Div., B-I-F Industries)
Foxboro Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Jos. G. Pollard Co., Inc.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Gasholders:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Gaskets, Rubber Packing:
James B. Clow & Sons
Johns-Manville Corp.

Gates, Shear and Sluice:
Armco Drainage & Metal Products, Inc.
Chapman Valve Mfg. Co.
James B. Clow & Sons
Morse Bros. Mchy. Co.
Mueller Co.
R. D. Wood Co.

Gears, Speed Reducing:
DeLaval Steam Turbine Co.
Philadelphia Gear Works, Inc.

Glass Standards—Colorimetric Analysis Equipment:
Klett Mfg. Co.
Wallace & Tiernan Inc.

Goosenecks (with or without Corporation Stops):
James B. Clow & Sons
Hays Mfg. Co.
Mueller Co.

Hydrants:
James B. Clow & Sons
Darling Valve & Mfg. Co.
M. Greenberg's Sons
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
Rensselaer Valve Co.
R. D. Wood Co.

Hydrogen Ion Equipment:
Wallace & Tiernan Inc.

Hypochlorite; see Calcium Hypochlorite; Sodium Hypochlorite

Ion Exchange Materials:
Allis-Chalmers Mfg. Co.
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.

Iron, Pig:
Woodward Iron Co.

Iron Removal Plants:
American Well Works
Chain Belt Co.
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Jointing Materials:
Hydraulic Development Corp.
Johns-Manville Corp.
Keasbey & Mattison Co.
Leadite Co., Inc.

Joints, Mechanical, Pipe:
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Dresser Mfg. Div.
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Leak Detectors:
Jos. G. Pollard Co., Inc.

Lime Slakers and Feeders:
Dorr-Oliver Inc.
General Filter Co.
Infilco Inc.
Omega Machine Co. (Div., B-I-F Industries)
Permutit Co.
Wallace & Tiernan Inc.

Magnetic Dipping Needles:
W. S. Darley & Co.

Meter Boxes:
Ford Meter Box Co.
Pittsburgh Equitable Meter Div.

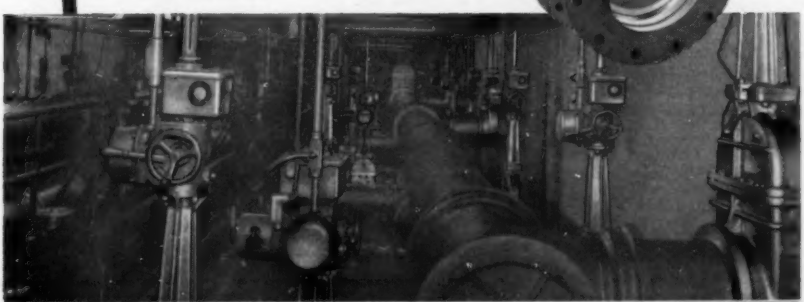
FILTER PLANT EQUIPPED WITH MOTOR OPERATED M-H VALVES

There are four principal uses for electric motor operated M & H Gate Valves:

1. **Facilitate opening and closing of large valves.**
2. **Control and expedite frequent operation of valves.**
3. **Simplify operation of valves installed in remote, inaccessible or hazardous locations.**
4. **Reduce operating costs and increase efficiency.**

The motor operated valves in the pipe gallery shown below are standard M & H Gate Valve design and construction. They are available in all sizes for 50, 100, 150 or 250 lbs. working pressure, and with standard accessories such as square bottom design, rollers-tracks-and-scrapers, indicators, etc. For complete information, address

M&H VALVE AND FITTINGS COMPANY
Anniston, Alabama



M&H PRODUCTS FOR WATER WORKS • FILTER PLANTS INDUSTRY • SEWAGE DISPOSAL AND FIRE PROTECTION

Meter Couplings and Yokes:
Badger Meter Mfg. Co.
Dresser Mfg. Div.
Ford Meter Box Co.
Hays Mfg. Co.
Hersey Mfg. Co.
Mueller Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Worthington-Gamon Meter Co.

Meter Reading and Record Books:
Badger Meter Mfg. Co.

Meter Testers:
Badger Meter Mfg. Co.
Ford Meter Box Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.

Meters, Domestic:
Badger Meter Mfg. Co.
Buffalo Meter Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Meters, Filtration Plant, Pumping Station, Transmission Line:
Builders-Providence, Inc. (Div., B-I-F Industries)
Foster Eng. Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Simplex Valve & Meter Co.

Meters, Industrial, Commercial:
Badger Meter Mfg. Co.
Buffalo Meter Co.
Builders-Providence, Inc. (Div., B-I-F Industries)
Fischer & Porter Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Simplex Valve & Meter Co.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Mixing Equipment:
Chain Belt Co.
General Filter Co.
Infilco Inc.
F. B. Leopold Co.

Paints:
Barrett Div.
Inertol Co., Inc.

Pipe, Asbestos-Cement:
Johns-Manville Corp.
Keasbey & Mattison Co.

Pipe, Brass:
American Brass Co.

Pipe, Cast Iron (and Fittings):
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Cement Lined:
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Concrete:
American Concrete Pressure Pipe Assn.
American Pipe & Construction Co.
Lock Joint Pipe Co.

Pipe, Copper:
American Brass Co.

Pipe, Steel:
Alco Products, Inc.
Armco Drainage & Metal Products, Inc.
Bethlehem Steel Co.

Pipe Cleaning Services:
Ace Pipe Cleaning, Inc.
National Water Main Cleaning Co.

Pipe Cleaning Tools and Equipment:
Flexible Inc.

Pipe Coatings and Linings:
Barrett Div.
Cast Iron Pipe Research Assn.
Centriline Corp.
Inertol Co., Inc.
Koppers Co., Inc.
Reilly Tar & Chemical Corp.

Pipe Cutters:
James B. Clow & Sons
Ellis & Ford Mfg. Co.
Jos. G. Pollard Co., Inc.
Reed Mfg. Co.
A. P. Smith Mfg. Co.
Spring Load Mfg. Corp.

Pipe Jointing Materials; see Jointing Materials

Pipe Locators:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pipe Vises:
Spring Load Mfg. Corp.

Plugs, Removable:
James B. Clow & Sons
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Potassium Permanganate:
Carus Chemical Co.

Pressure Regulators:
Allis-Chalmers Mfg. Co.
Foster Eng. Co.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Ross Valve Mfg. Co.

Pumps, Boiler Feed:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Pumps, Centrifugal:
Allis-Chalmers Mfg. Co.
American Well Works
DeLaval Steam Turbine Co.
Morse Bros. Mchy. Co.
C. H. Wheeler Mfg. Co.

Pumps, Chemical Feed:
Infilco Inc.
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

Pumps, Deep Well:
American Well Works
Layne & Bowler, Inc.

Pumps, Diaphragm:
Dorr-Oliver Inc.
Morse Bros. Mchy. Co.
Wallace & Tiernan Inc.

Pumps, Hydrant:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pumps, Hydraulic Booster:
Ross Valve Mfg. Co.

Pumps, Sewage:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
C. H. Wheeler Mfg. Co.

Pumps, Sump:
DeLaval Steam Turbine Co.
C. H. Wheeler Mfg. Co.

Pumps, Turbine:
DeLaval Steam Turbine Co.
Layne & Bowler, Inc.

Recorders, Gas Density, CO_2, NH_3, SO_2, etc.:
Permutit Co.
Wallace & Tiernan Inc.

Recording Instruments:
Builders-Providence, Inc. (Div., B-I-F Industries)
Fischer & Porter Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Reservoirs, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Sand Expansion Gages; see Gages

Sleeves; see Clamps

Sleeves and Valves, Tapping:
James B. Clow & Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Sludge Blanket Equipment:
General Filter Co.
Graver Water Conditioning Co.
Permutit Co.

Sodium Aluminate:
Monolith Portland Midwest Co.

Sodium Chloride:
Frontier Chemical Co.

Sodium Hexametaphosphate:
Calgon, Inc.

Sodium Hypochlorite:
John Wiley Jones Co.
Wallace & Tiernan Inc.

Sodium Silicate:
Philadelphia Quartz Co.

Softeners:
Cochrane Corp.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
Calgon, Inc.
Cochrane Corp.
General Filter Co.
Infilco Inc.
Morton Salt Co.
Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

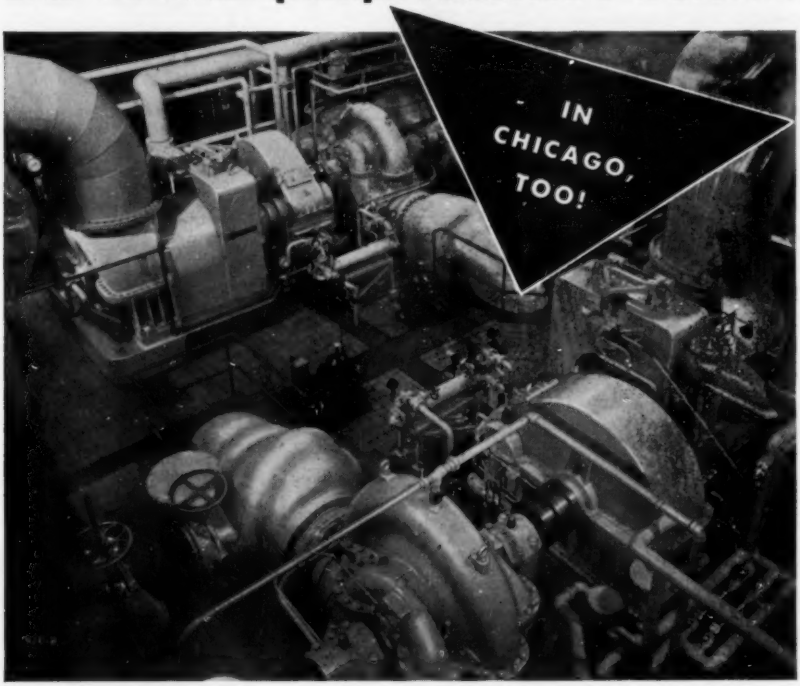
Steel Plate Construction:
Alco Products, Inc.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Stops, Curb and Corporation:
Hays Mfg. Co.
Mueller Co.

Storage Tanks; see Tanks

Strainers, Suction:
James B. Clow & Sons
M. Greenberg's Sons
Johnson, Edward E., Inc.
R. D. Wood Co.

DE LAVAL pumps America's water...



The two De Laval centrifugal pumps shown in the photograph are on the job in the Mayfair station of the Chicago water works system. They each have a rated capacity of 60 million gallons per day. De Laval has supplied 28 units for the various Chicago stations with a total rated capacity of 1695 million gallons per day. Two additional units are on order which will bring the capacity to 1825 mgd in the near future.

Today, 80% of the cities in the United States with a population of 100,000 or over use De Laval centrifugal pumps. *Write for your copy of new De Laval Bulletin 1004 giving data on these pumps.*

MR. JAMES W. JARDINE
Commissioner
Department of Water & Sewers

MR. W. W. DEBERARD
Deputy Commissioner for Water and Chief Water Engineer

MR. O. B. CARLISLE
Chief Water Works Engineer
Department of Public Works

MR. JAMES L. WEEKS
Mechanical Engineer in Charge Operating Division

DE LAVAL *Centrifugal Pumps*

DE LAVAL STEAM TURBINE COMPANY

822 Nottingham Way, Trenton 2, New Jersey

DL326A

Surface Wash Equipment:
Cochrane Corp.
Permutit Co.

Swimming Pool Sterilization:
Builders-Providence, Inc. (Div., B-I-F Industries)
Fischer & Porter Co.
Omega Machine Co. (Div., B-I-F Industries)
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

Tanks, Steel:
Alco Products, Inc.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Tapping-Drilling Machines:
Hays Mfg. Co.
Mueller Co.
A. P. Smith Mfg. Co.

Tapping Machines, Corp.:
Hays Mfg. Co.
Mueller Co.

Taste and Odor Removal:
Builders-Providence, Inc. (Div., B-I-F Industries)
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Industriai Chemical Sales Div.
Infilco Inc.
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

Tenoning Tools:
Spring Load Mfg. Corp.

Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Wallace & Tiernan Inc.

Turbines, Steam:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Turbines, Water:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Valve Boxes:
James B. Clow & Sons
Ford Meter Box Co.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.
R. D. Wood Co.

Valve-Inserting Machines:
Mueller Co.
A. P. Smith Mfg. Co.

Valves, Altitude:
Golden-Anderson Valve Specialty Co.
Ross Valve Mfg. Co., Inc.
S. Morgan Smith Co.

Valves, Butterfly, Check, Flap, Foot, Hose, Mud and Plug:
Builders-Providence, Inc. (Div., B-I-F Industries)
Chapman Valve Mfg. Co.
James B. Clow & Sons
DeZurik Shower Co.
M. Greenberg's Sons
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Detector Check:
Hersey Mfg. Co.

Valves, Electrically Operated:
Builders-Providence, Inc. (Div., B-I-F Industries)
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Philadelphia Gear Works, Inc.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.

Valves, Float:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Henry Pratt Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
Builders-Providence, Inc. (Div., B-I-F Industries)
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
DeZurik Shower Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
F. B. Leopold Co.
M & H Valve & Fittings Co.
Mueller Co.
Philadelphia Gear Works, Inc.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Large Diameter:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Regulating:
DeZurik Shower Co.
Foster Eng. Co.
Golden-Anderson Valve Specialty Co.
Minneapolis-Honeywell Regulator Co.
Mueller Co.
Henry Pratt Co.
Ross Valve Mfg. Co.
S. Morgan Smith Co.

Valves, Swing Check:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
M. Greenberg's Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Venturi Tubes:
Builders-Providence, Inc. (Div., B-I-F Industries)
Infilco Inc.
Simplex Valve & Meter Co.

Waterproofing:
Barrett Div.
Inertol Co., Inc.

Water Softening Plants; see Softeners

Water Supply Contractors:
Layne & Bowler, Inc.

Water Testing Apparatus:
Wallace & Tiernan Inc.

Water Treatment Plants:
American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Cochrane Corp.
Dorr-Oliver Inc.
Fischer & Porter Co.
General Filter Co.
Graver Water Conditioning Co.
Hammond Iron Works
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.

Well Drilling Contractors:
Layne & Bowler, Inc.

Well Screens:
Johnson, Edward E., Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite; see Ion Exchange Materials

A complete Buyers' Guide to all water works products and services offered by AWWA Associate Members appears in the 1955 AWWA Directory.

YOU WERE DOING THIS

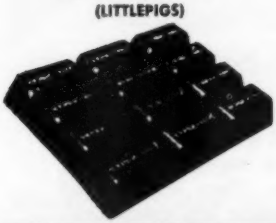
WHEN THE FIRST HYDRO-TITE JOINTS WERE BEING POURED -

HYDRO-TITE (POWDER)

HYDRO-TITE (POWDER)

For over 40 years HYDRO-TITE has been faithfully serving water works men everywhere. Self-caulking, self-sealing, easy-to-use. Costs about 1/5 as much as lead joints. Packed in 100 lb. moisture-proof bags.

HYDRO-TITE (LITTLEPIGS)



HYDRO-TITE (LITTLEPIGS)

The same dependable compound in solid form —packed in 50 lb. cartons—2 litters of pigs to the box—24 easy-to-handle Littlepigs. Easier to ship, handle and store.

FIBREX (REELS)

FIBREX (REELS)

The sanitary, bacteria-free joint packing. Easier to use than jute and costs about half as much. Insures sterile mains and tight joints.

HYDRO-TITE

HYDRAULIC DEVELOPMENT CORPORATION

TWO HEADS ARE BETTER THAN FOUR!

This assembly consists of two 4-in Rockwell single register compound meters, four Fig. 143 Nordstrom valves and two 8-in reducing manifolds. The over-all laying length conforms to A.W.W.A. standards for 8-in compound meters

Dual Unit Compound Meter Manifold Is Easiest To Install and Service

An 8-in compound meter is a heavy, bulky piece of machinery. Installation is always a major problem and service a headache. You'll save time and effort—earn greater revenue by using two-meter Rockwell manifold assemblies. *They measure all the flows* with greater accuracy than a single big compound. And the complete assembly weighs approximately 100 lb less. It only takes *two* men to install.

Too, maintenance is simplified since either meter can be used to record off-peak loads while a new or repaired meter is being installed. The cost for all this accuracy and convenience is usually less than for a single big meter. Write for complete details.

ROCKWELL MANUFACTURING COMPANY

PITTSBURGH 8, PA. Atlanta Boston Charlotte Chicago Dallas Denver Houston Los Angeles Midland, Tex. New Orleans New York N. Kansas City Philadelphia Pittsburgh San Francisco Seattle Shreveport Tulsa

In Canada: Rockwell Manufacturing Company of Canada, Ltd., Toronto, Ontario

ROCKWELL WATER METERS

Water Treatment, too, needs

INDIVIDUAL DIAGNOSIS

No two water treatment problems are exactly alike. The right solution to each can only be arrived at after a careful study of the local conditions. Variables such as raw water composition, rate of flow and results required automatically rule out the cure-all approach. The installation shown below is a good example of how equipment should be selected to fit the job . . . and not vice versa.

"Plantation Gardens"
Florida

DORRCO
ALDRICH
PeriFilter* System

Meets Filtration Requirements of 1000 Home Subdivision

Consulting Engineers. Philpott, Ross & Saarinen, Ft. Lauderdale, Florida
Owner-Builder. Utilities Construction Company, Ft. Lauderdale, Florida

Plantation Gardens, an exclusive subdivision of 1000 homes having a population of 3500 persons, recently started up this compact water treatment plant for softening and color removal.

A Dorrco Aldrich PeriFilter System, consisting of a Dorrco Hydro-Treator surrounded by an annular rapid sand filter, was selected as the most economical answer to meet the conditions. The plant is designed to handle an average flow of 0.53 MGD with plans for enlargement at a future date.

Advantages of the PeriFilter system include lower construction costs because both pre-treatment unit and filter are installed in the same tank. Valves and piping are greatly simplified. Reduced head losses and simple operation add up to lower operating costs.

If you'd like more information on the PeriFilter System write for Bulletin No. 9042. No obligation, of course.

Hydro-Treator, PeriFilter T.M. Reg. U. S. Pat. Off.

Every day, nearly 8½ billion gallons of water are treated with Dorr-Oliver equipment.

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WORLD-WIDE RESEARCH • ENGINEERING • EQUIPMENT
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LEADITE

Trade Mark Registered U. S. Pat. Office

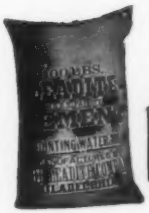
Jointed for . . . Permanence with LEADITE

Generally speaking, most Water Mains are buried beneath the Earth's surface, to be forgotten,—they are to a large extent, laid for permanency. Not only must the pipe itself be dependable and long lived,—but the joints also must be tight, flexible, and long lived,—else leaky joints are apt to cause the great expense of digging up well-paved streets, beautiful parks and estates, etc.

Thus the "jointing material" used for bell and spigot Water Mains MUST BE GOOD,—MUST BE DEPENDABLE,—and that is just why so many Engineers, Water Works Men and Contractors aim to PLAY ABSOLUTELY SAFE, by specifying and using LEADITE.

Time has proven that LEADITE not only makes a tight durable joint,—but that it improves with age.

The pioneer self-caulking material for c. i. pipe.
Tested and used for over 40 years.
Saves at least 75%



THE LEADITE COMPANY
Girard Trust Co. Bldg. Philadelphia, Pa.